CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

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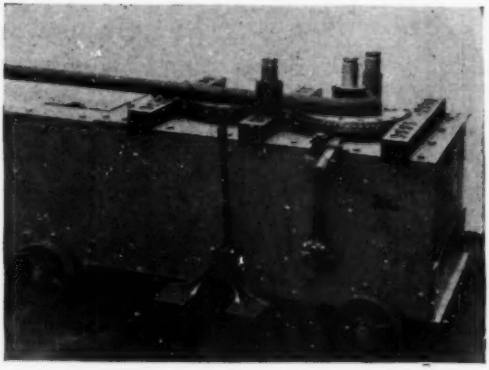

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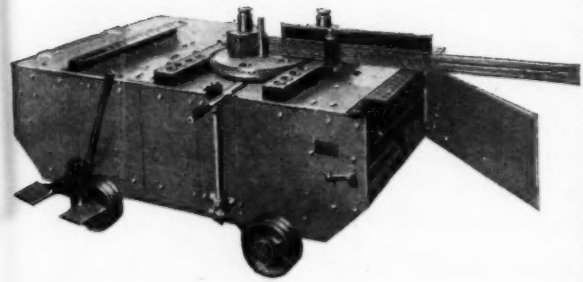

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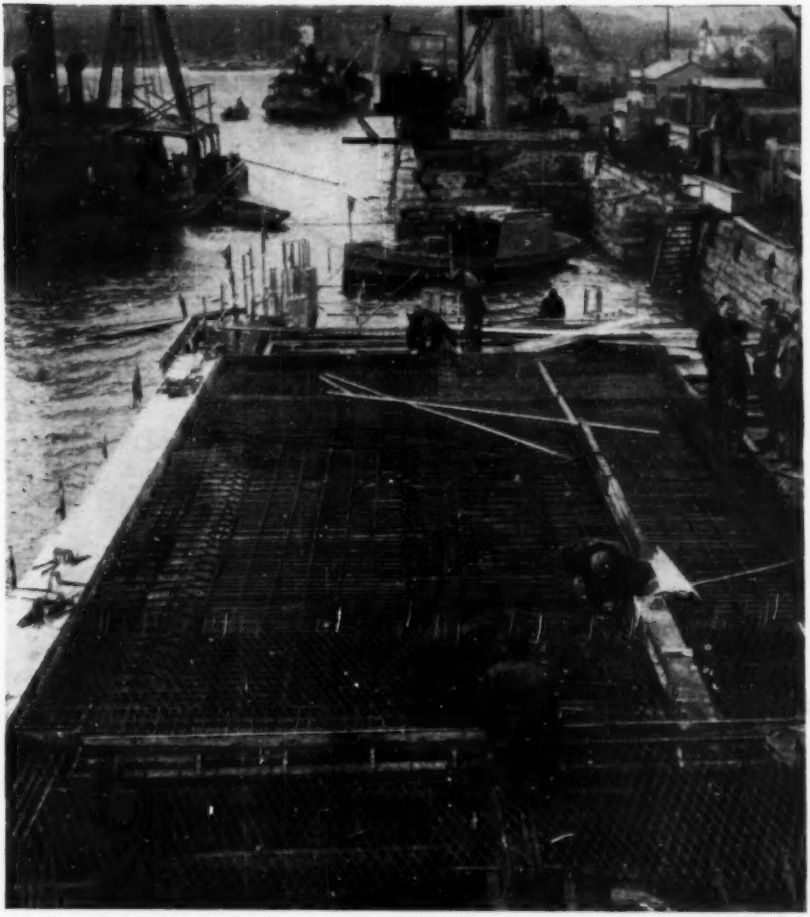

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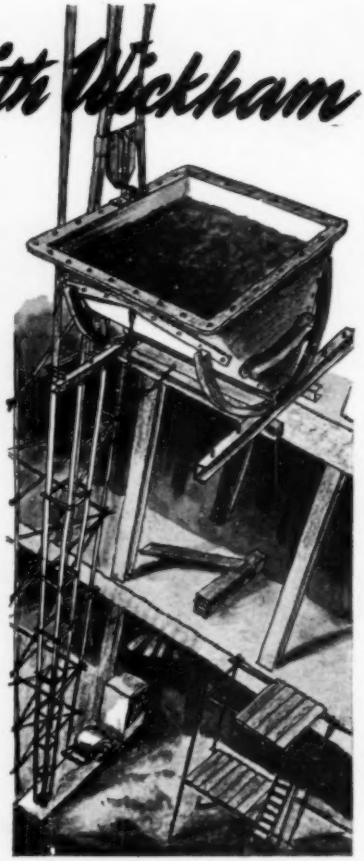

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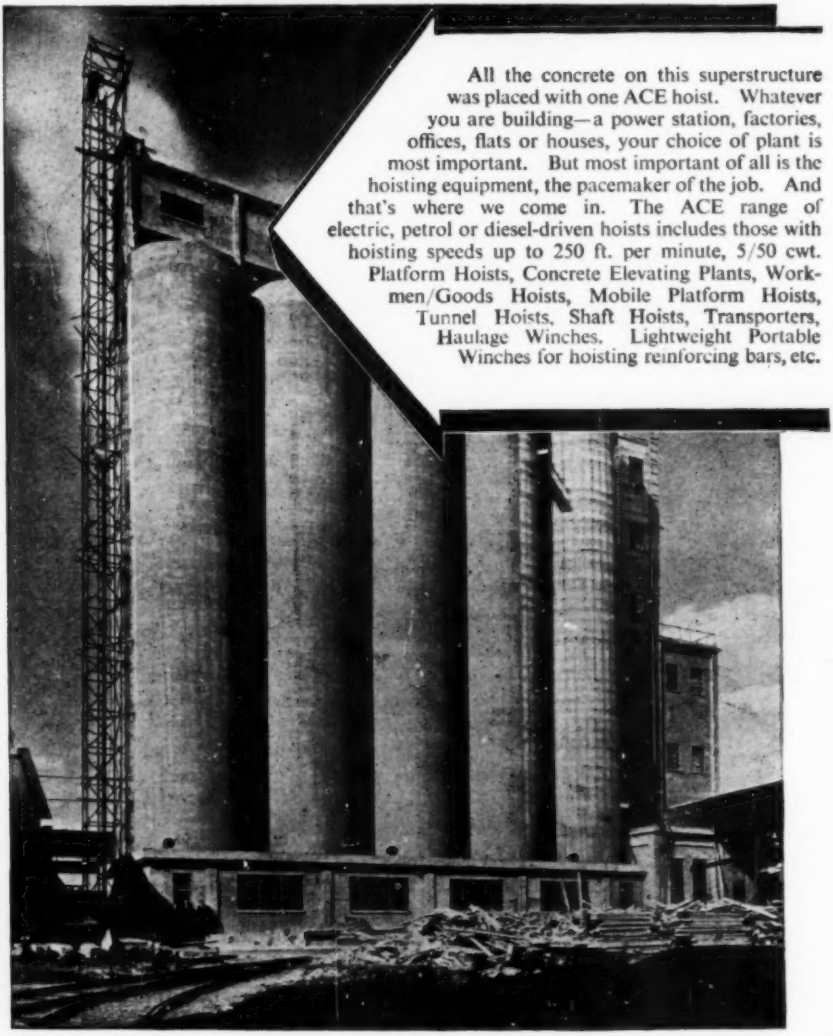

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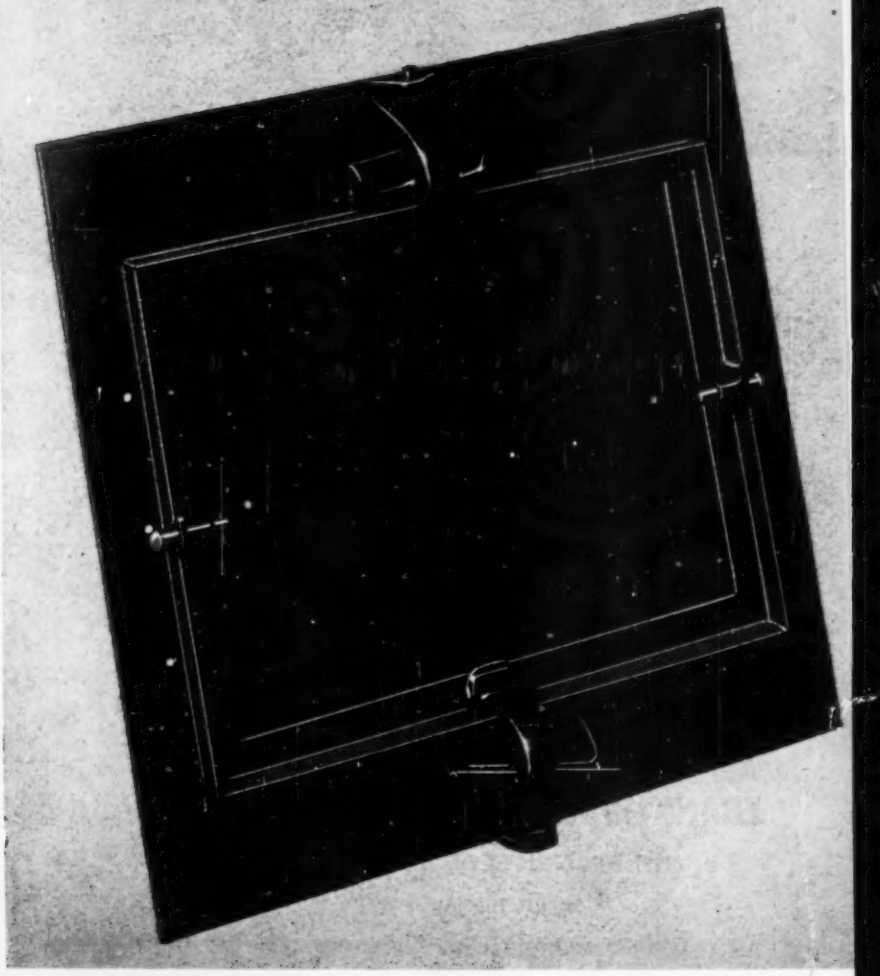

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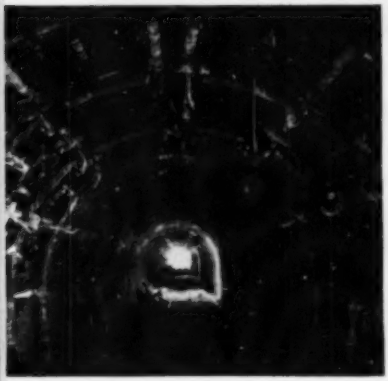

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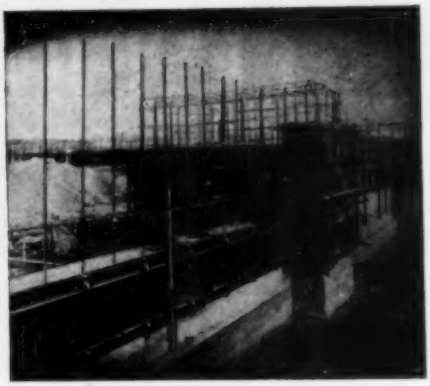

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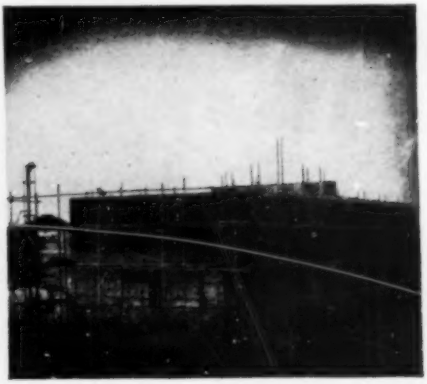

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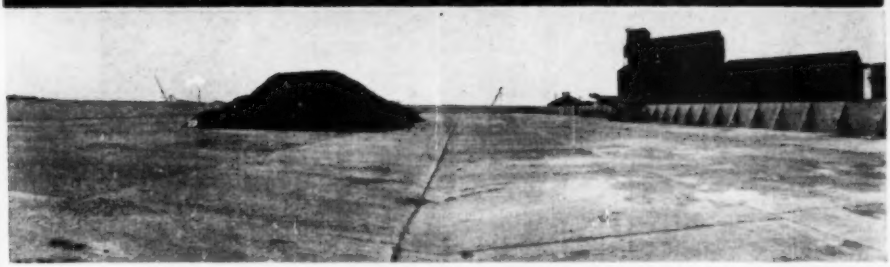

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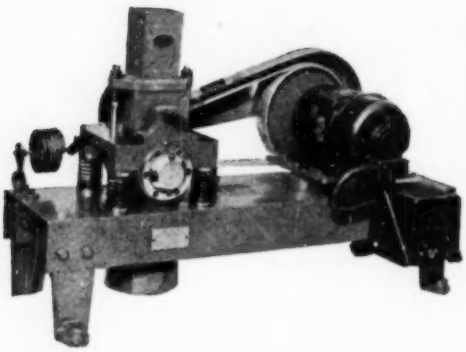

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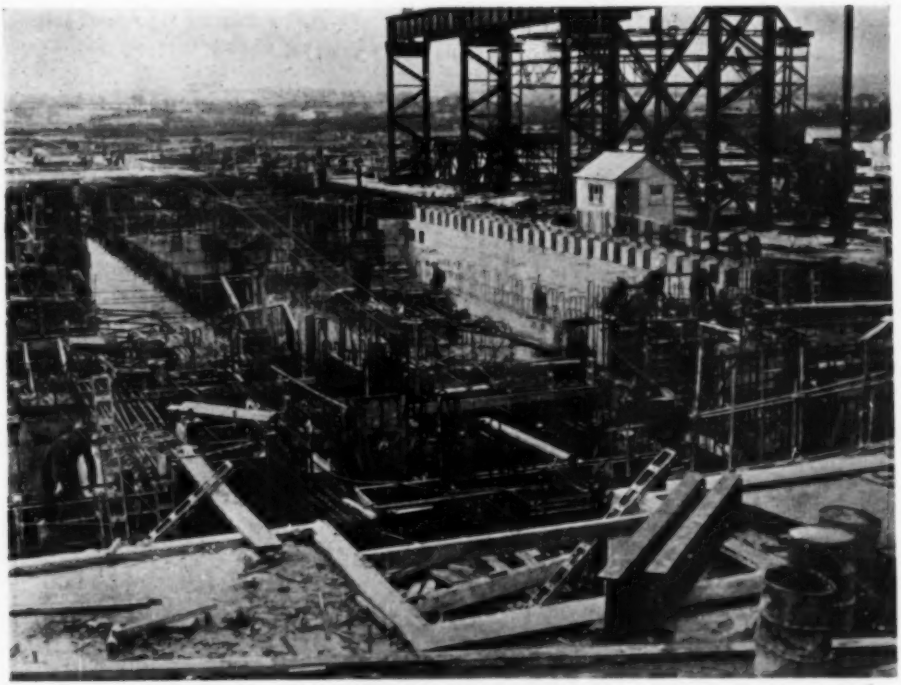

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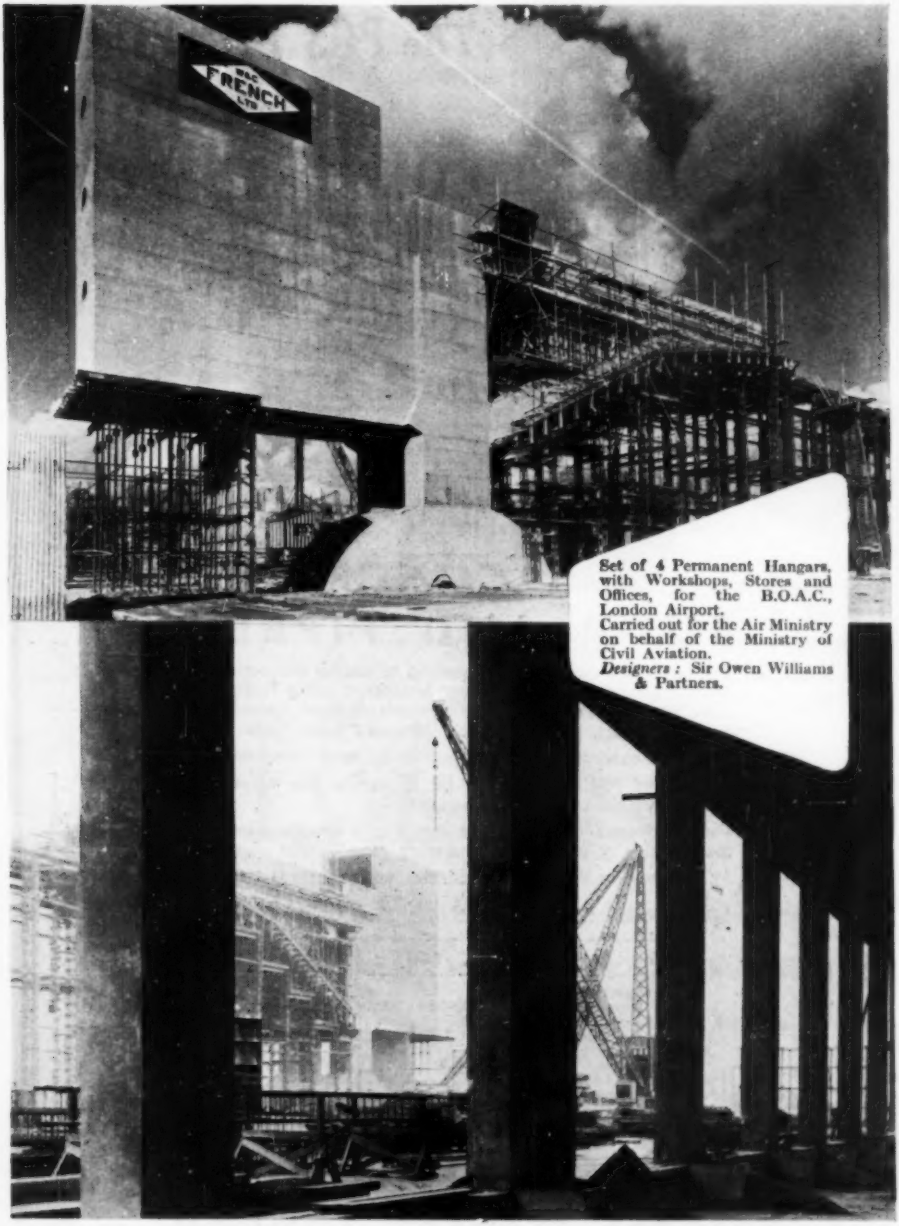

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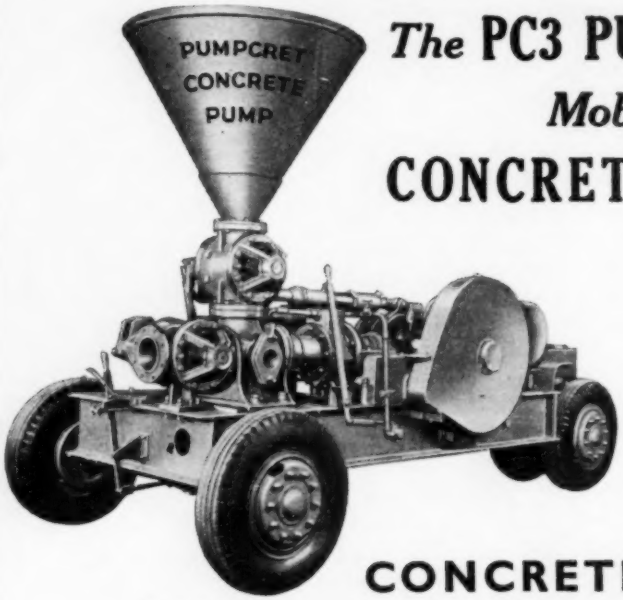

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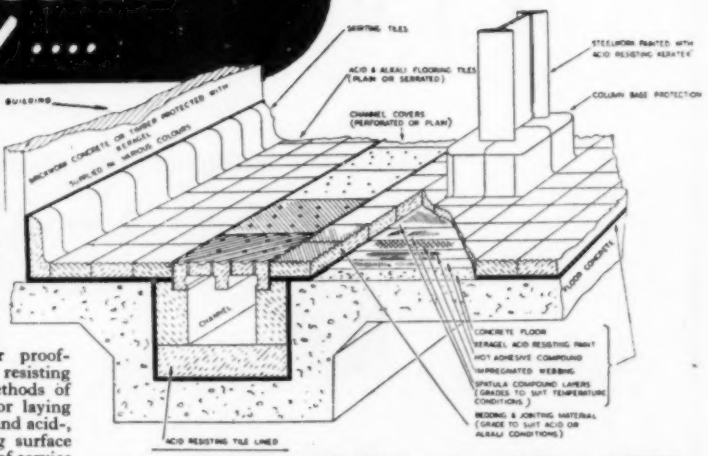
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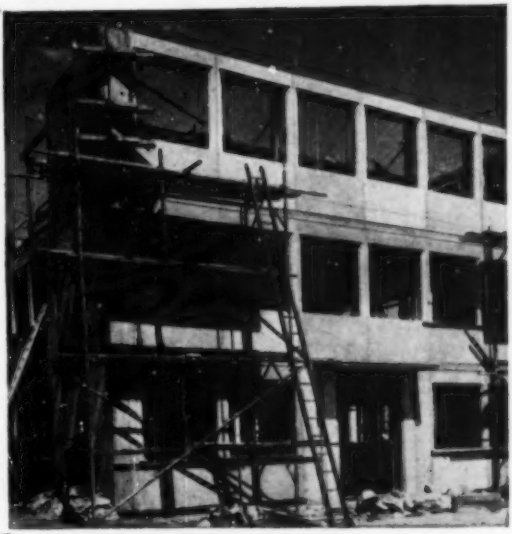

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Photograph by kind permission of the Yiewsley and West Drayton U.D.C. (Engineer, Surveyor and Architect, W. T. Morgan, F.R.I.C.S., M.I.Mun.E., L.R.I.B.A.), Town Hall, West Drayton, Middlesex.

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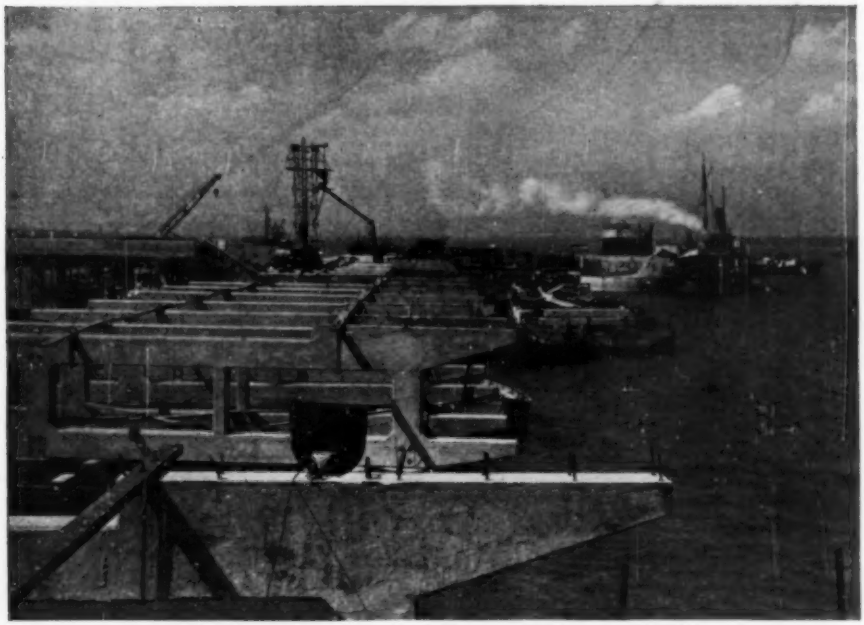

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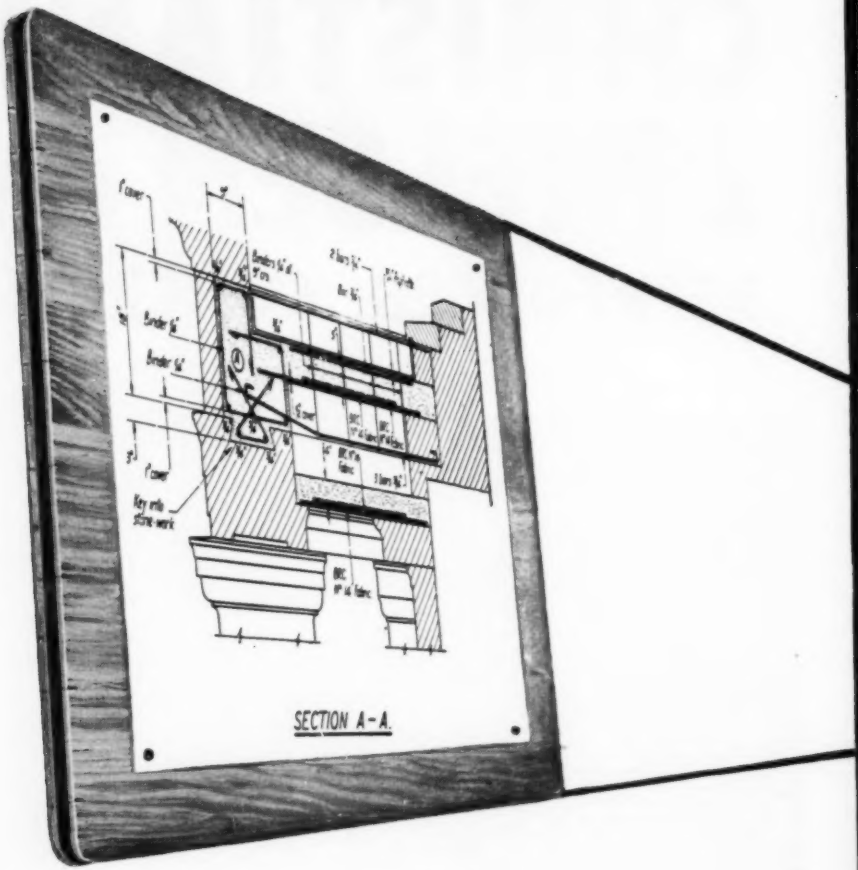

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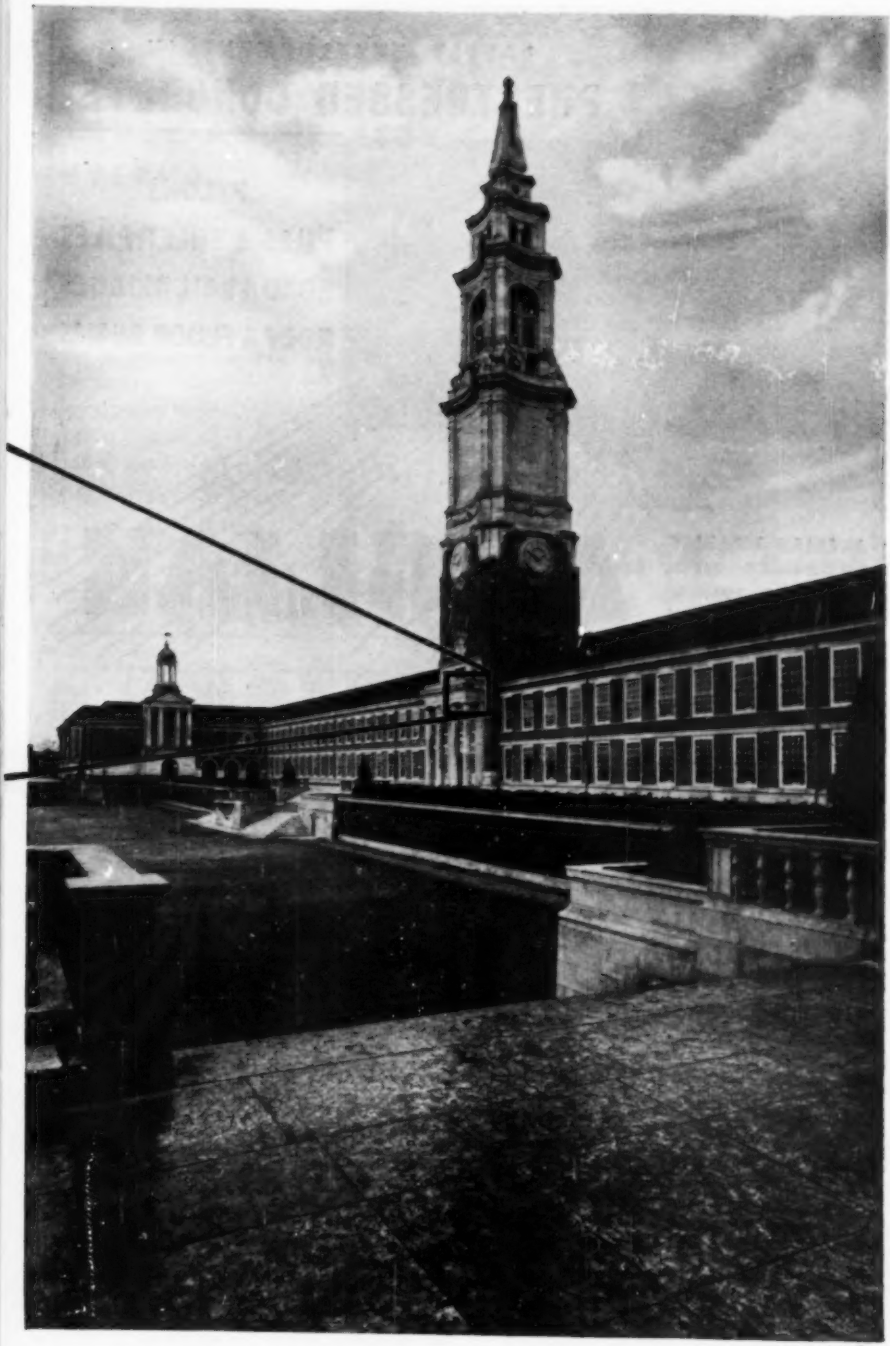

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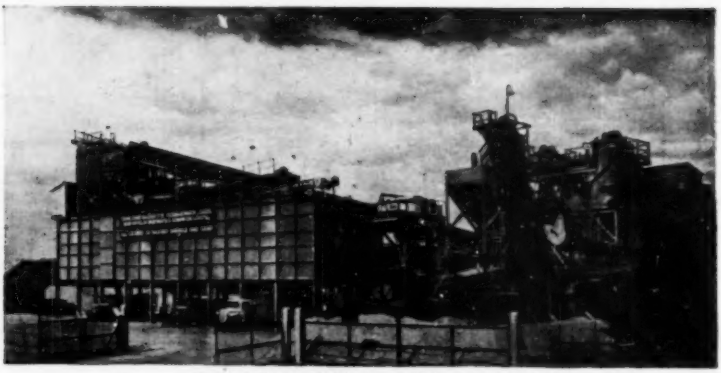

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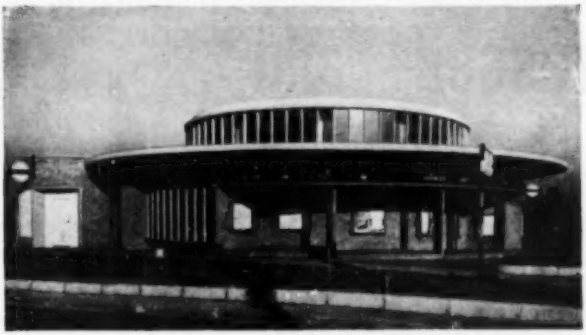

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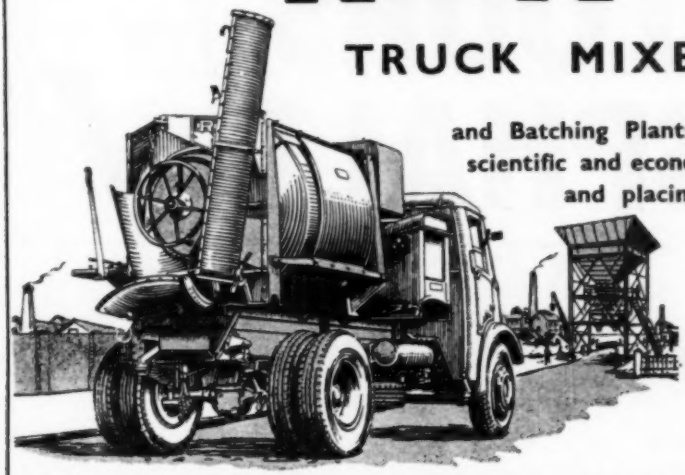

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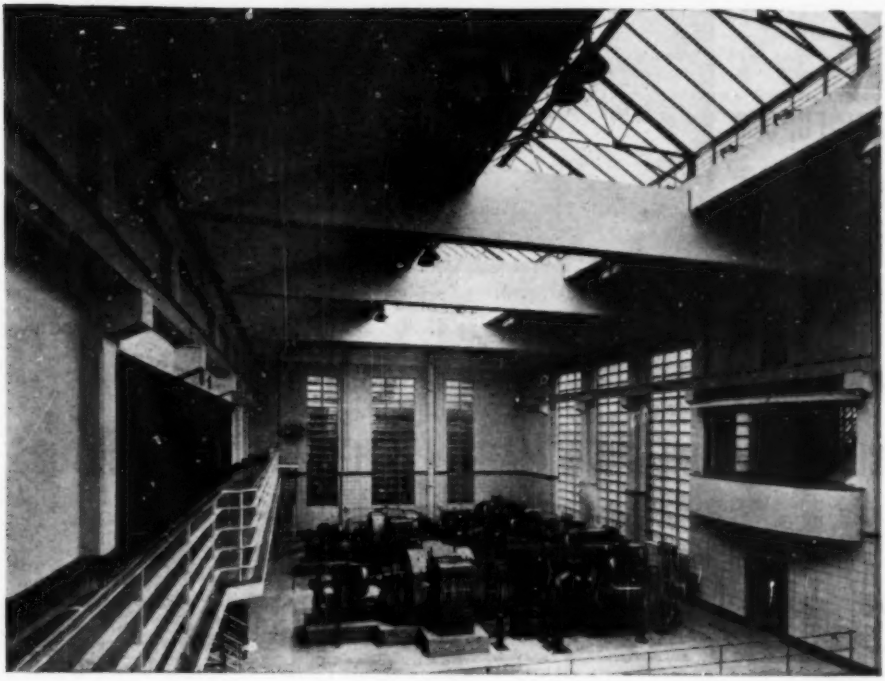

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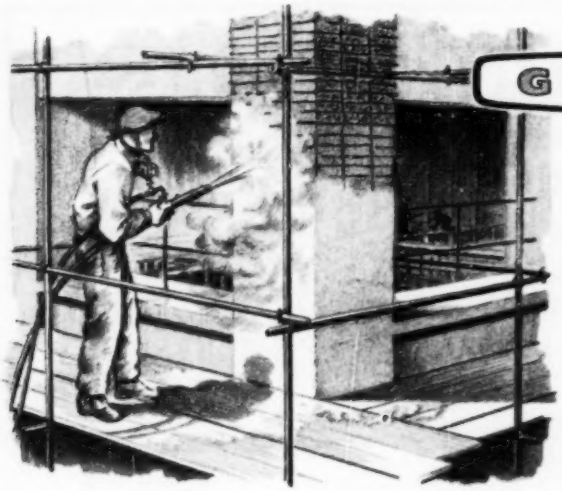

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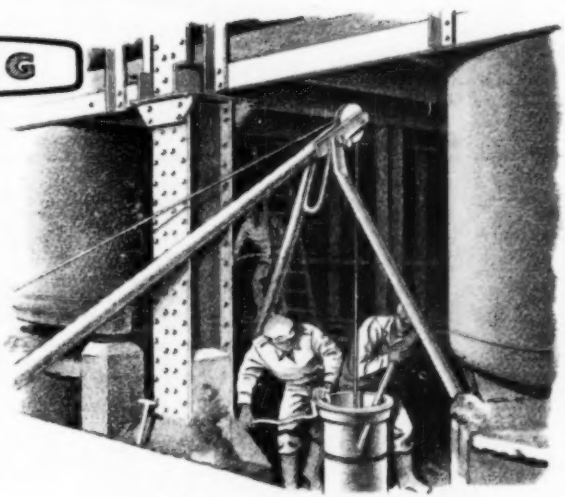

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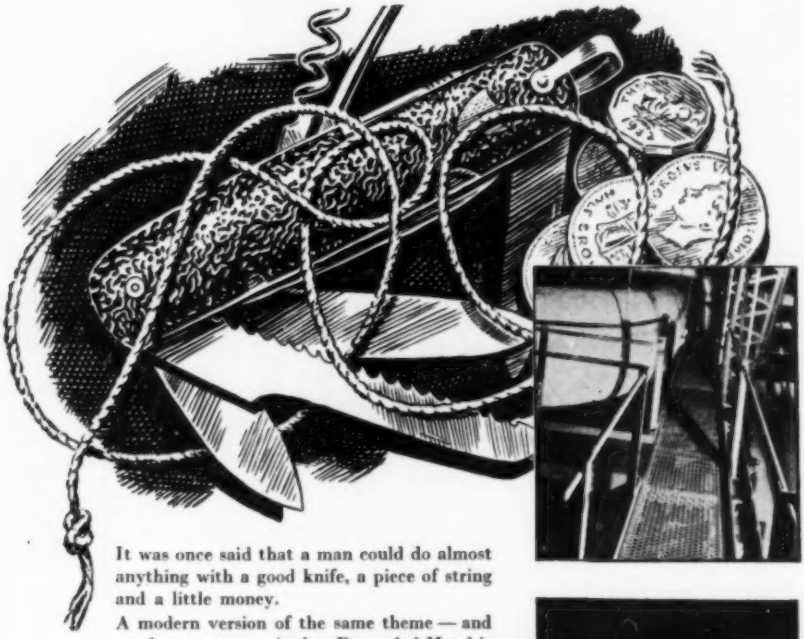

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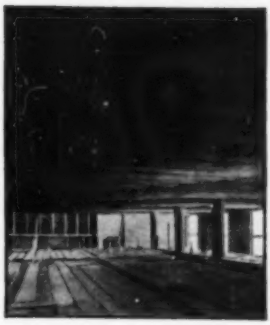

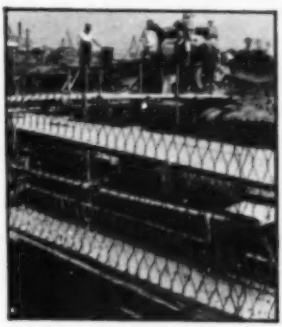

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● *Photographs illustrate (Top) Expanded Metal walkways in the Royal Festival Hall. (Centre) "BB" Lathing ceiling at Bow Road Methodist Church, London. Architects: Messrs. Mauger & May, F/F.R.I.B.A. (Bottom) Reinforcement for concrete—Electricity Sub-station, Brentford, for the Southern Electricity Board.*

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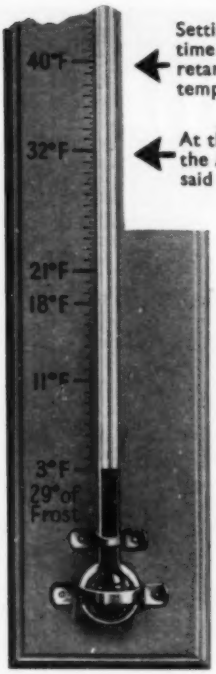

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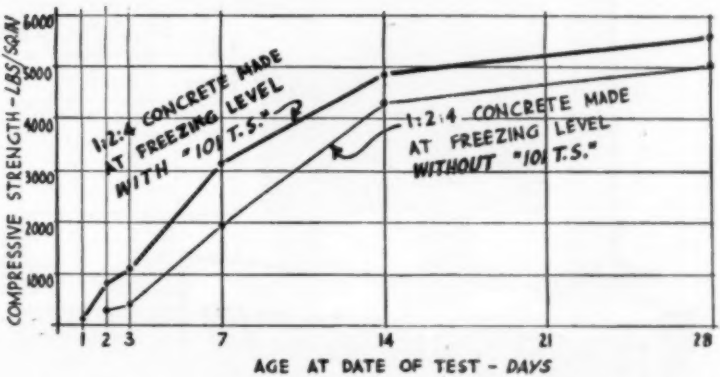

This chart shows the results of tests by R. H. Harry Stanger, Testing Engineer, of Westminster. After only one day, 6" test cubes of 1 : 2 : 4 concrete, made with 101 T.S., show measurable compressive strength. Concrete made without is too weak to be tested at all. For all times of test, 101 T.S. gives a permanent increase of strength over concrete made without. Formwork can be struck and structural concrete can be loaded as in normal weather—progress need not be interrupted.

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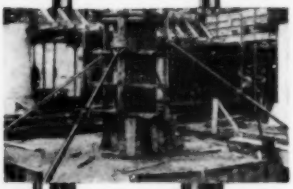
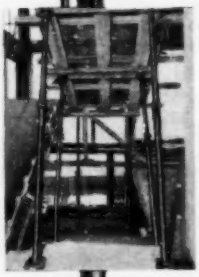
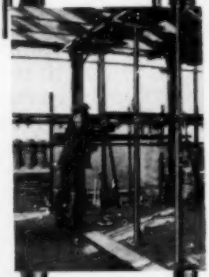
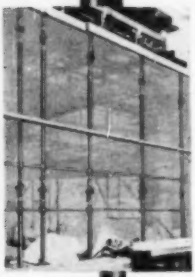
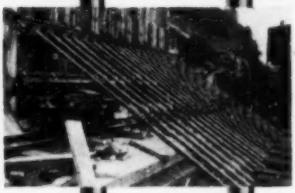

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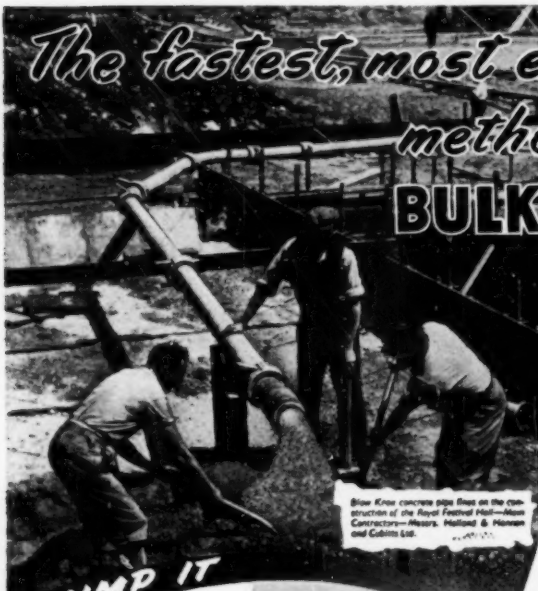

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES

Science in the Building Industry.

IT is not likely that all will agree with all of the views on the building industry expressed by the secretary of the Department of Scientific and Industrial Research in his address at the closing session of the Building Research Congress held recently in London. One of the sub-headings in this address as circulated by the Department is " Building Industry Unwilling to Learn ", the meaning of which is not altered in its context, which is " Building is now, to one degree or another, unwilling or unable to learn ", " the general picture is one of relative stagnation ", and " the building industry has allowed itself to become marooned ". If this were literally true there could be no better reason for closing the Building Research Station, which has been in existence for about thirty years and is now costing about half a million pounds a year. But this suggestion that it is futile to undertake research in building was contradicted by the speaker when he said that during this period improvements in building materials and components and a further understanding of their proper use had influenced design, construction, and maintenance ; and, to take one example only, that the cost of schools had been reduced from £195 per pupil to £140 per pupil during the past two years, presumably without reduction of efficiency or comfort.

One of the chief reasons why some builders may have been reluctant to invest large sums of money in plant has been the Government's changes of policy in the expenditure on building under successive economic plans. It is not likely that, when building work is restricted and the future cannot be foreseen, a builder will accumulate plant and have it ready for unexpected armament or other building programmes.

Some of the comparisons of increased production due to the use of machinery are also likely to be misleading unless they are related to Government programmes and the nature of different industries. For example, it is stated that 20 per cent. more workers have produced 39 per cent. greater output in agriculture in the past fourteen years, while in the building of houses for local authorities the output has fallen by 20 per cent. since 1939. We would suggest that no comparison is possible between agriculture, which has been consistently encouraged by subsidies, guaranteed prices, and other means to produce all it can, and house-building which has been restricted and is frequently delayed by lack of materials. Men cannot be expected to give of their best when they can see enough materials on the site to keep them employed for a few hours or days only, and know from

experience that these materials may be used long before a further supply is delivered. Also the tea breaks and other excuses for not working are a much greater factor in building than on the land. The address placed much importance on what is called operational research and " powerful methods of statistical mathematics ", which have been applied to some workers in the building trade to find out how much of their time is spent in useful work and how much is wasted. The result confirms what is generally known, but gives no guidance on how men are to be encouraged to work harder. This is a human problem which cannot be solved by such means. The man with the stop-watch can only record the time he sees wasted, and no amount of " statistical mathematics ", however powerful, will persuade a man to work harder if he does not wish to do so in times when he can get a living wage without giving a fair return in work.

The paper stated that in 1935 the building industry used half a horse-power per man, and this is compared with 2½ h.p. per man for all manufacturing industries. We would suggest that no useful comparison can be made between an industry employed on erection in the open air and manufacturing in factories. When we see the large amount of mechanisation, the standardisation, and the substitute materials, that are now used in building, it is hardly fair to say that the industry is unwilling to learn. More should be said about the discouragements with which the builder is faced, such as savings due to the use of machinery being continually counteracted by increased costs, the small depreciation allowed on plant for income-tax purposes which often makes it cheaper to hire plant for the whole of its working life than to own it, the difficulty of getting men who will do a fair day's work, and so on.

One of the easiest ways of reducing the cost of buildings is to make them flimsier by increasing the permissible design stresses in materials and so lowering the factor of safety. On this the speaker said that there is little doubt that we can economise in materials and reduce costs by reducing what he called over-insurance in design, and that a " large experimental effort " should be made to study the quality control of concrete. Two points may be made on these suggestions. First, the codes of practice on which design is generally based are prepared by experienced engineers and scientists (some of whom may be from the Building Research Station) under the aegis of the Ministry of Works, and if the stresses, loads, and rules recommended in these codes are considered to be those that can safely be adopted it follows that any increase in stresses or lowering of factors of safety will result in weaker structures, with the natural consequences. Secondly, of the two materials used in reinforced concrete, steel is generally accepted as being of uniform quality, and means of making stronger and more uniform concrete have been and are now being extensively studied not only at the Building Research Station but in nearly every country in the world. Codes of practice are altered and higher stresses are recommended when new or improved materials are available, or more knowledge is gained of how buildings and materials work. These are within the province of research organisations, and on the results of their work depend whether higher stresses can be used and the present factors of safety maintained. A reduction of factors of safety which are generally considered necessary is not the best way of saving cost and labour. It is better for the scientists to show industry how to produce stronger materials and use less of them.

Large Reservoir Designed for Mining Subsidence.

THE Redhill service reservoir, in course of construction (*Fig.* 1) for the Corporation of Nottingham, will have a capacity of 4,000,000 gallons and is designed to conform to ground movements and to reduce the stresses due to severe mining subsidence. This is necessary because of the existence of unworked seams of coal below the site, and no other site in the district is free from the same problem.

Effect of Mining Operations.

The average depth below surface and the thickness of the four seams are: (1) 335 yd. deep, 4 ft. 4 in. thick; (2) 445 yd. deep, 3 ft. 3 in. thick; (3) 460 yd. deep, 2 ft. 5 in. thick; (4) 537 yd. deep, 3 ft. 6 in. thick. The bottom seam has been worked and the movements of the ground recorded. Mining of the top seam is proceeding, and it is anticipated that disturbance of the ground below the reservoir will commence in the middle of 1952 and will continue for several years. From the point at which mining operations are expected first to cause movement under the reservoir, the length of the coal face will be extended to the limits of the area of disturbance on each side of the reservoir, and mining will continue below and beyond the reservoir to the point at which disturbance will cease. The rate of cutting in the area will be about 40 yd. per month. From observations it is expected that the vertical settlement of the ground at the surface will be about 3 ft., that there will be considerable lateral pull, and that the mean gradient of differential settlement will be about 1 in 180.

Although the angle (on plan) of the coal face to the axis of the reservoir is known in the case of the top seam, similar information is not yet available with respect to the two intermediate seams. It is assumed therefore that future mining operations may approach the reservoir from any angle, an assumption that has an important bearing upon the design.

If the character and sequence of the movements of the ground be known, the behaviour of a rigid structure on the ground can be visualised. The surface movements are such that a structure bearing initially continuously on the ground may be subjected alternately to sagging and hogging effects. Therefore, for a reservoir of known depth, the greatest size of an economical structural unit which can safely resist the resulting stresses can be determined. It might appear that storage of the required capacity could be provided by a number of smaller reservoirs, each of the economical size determined, and the pipes and valves arranged to permit operation either individually or together. Investigation in this case, however, showed that a single reservoir of the design described in the following is much less costly and can be accommodated on a smaller area than several smaller reservoirs, and there are also operational advantages.

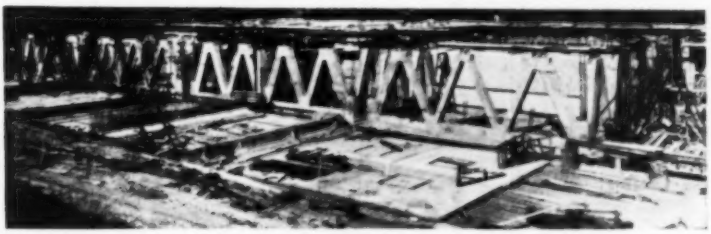


Fig. 1.—Reservoir in Course of Construction.

Design.

The depth of the reservoir is about 16 ft. and the mean depth of water 13 ft. The internal dimensions on plan are 230 ft. by 220 ft. For convenience of operation, the reservoir is divided by longitudinal

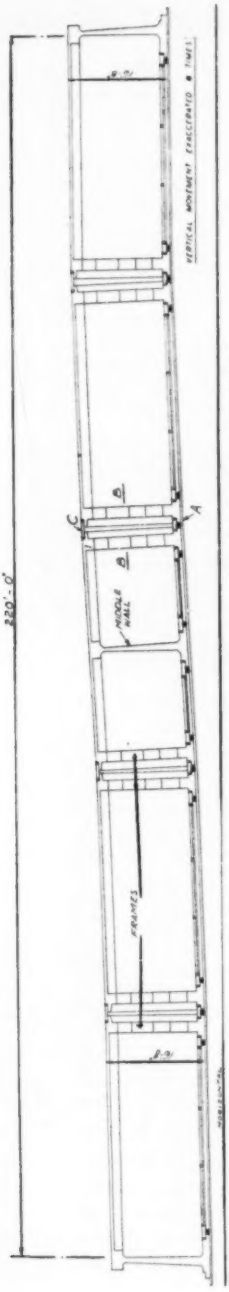


Fig. 2.—Assumed Deformation. (See Fig. 3 for Details.)

and transverse walls into four equal compartments. The reservoir is also divided into twenty-five independent structural units each about 46 ft. by 44 ft. Each rectangular unit comprises part of the floor and roof, and is bounded along its sides either by walls or lattice girders, which are designed as deep beams to resist ground movements as well as water pressure, dead weight, and the load on the roof. The units are connected only at floor level, where hinges are provided about which each unit can rotate. There is a gap between adjacent units to permit freedom of movement between the walls and roof. All pipes within the reservoir have flexible joints capable of accommodating the greatest expected movement.

Fig. 2 is a cross-section of the reservoir and illustrates the character and extent of movement that can be expected and the manner in which the jointed structure allows for the movement. *Fig.* 3 shows details (C) of the free joint in the roof, a horizontal section (B—B) through a watertight joint in the wall, and details (A) of a hinged joint in the floor, which is also shown in *Fig.* 5. A joint in the floor and the corresponding joint through the wall with the girders on either side are shown in *Fig.* 4. The hinged joints in the floor provide an anchorage between the units to enable the structure as a whole to resist the lateral pull. The gaps in the walls are 4 in. wide when constructed and are made watertight by cover-plates of sheet lead weighing 10 lb. per square foot. At each joint in the wall there is an external inspection chamber as shown on the sectional plan in *Fig.* 3. A ground drain is provided around the reservoir immediately outside the walls. The gaps in the roof are covered with precast concrete slabs, bedded and jointed in a plastic composition.

In view of the high bearing value of the ground at foundation level, a 10-in. bed of clay puddle is provided over the whole area of the excavation, and is covered with 4 in. of plain concrete to form a level slab upon which the structure is founded. The plastic bed of clay reduces considerably the stresses resulting from sagging and hogging movements, and limits the pull transmitted from the ground. It is assumed that the clay does not yield under a pressure of about 1 ton per square foot, but that when movement occurs it

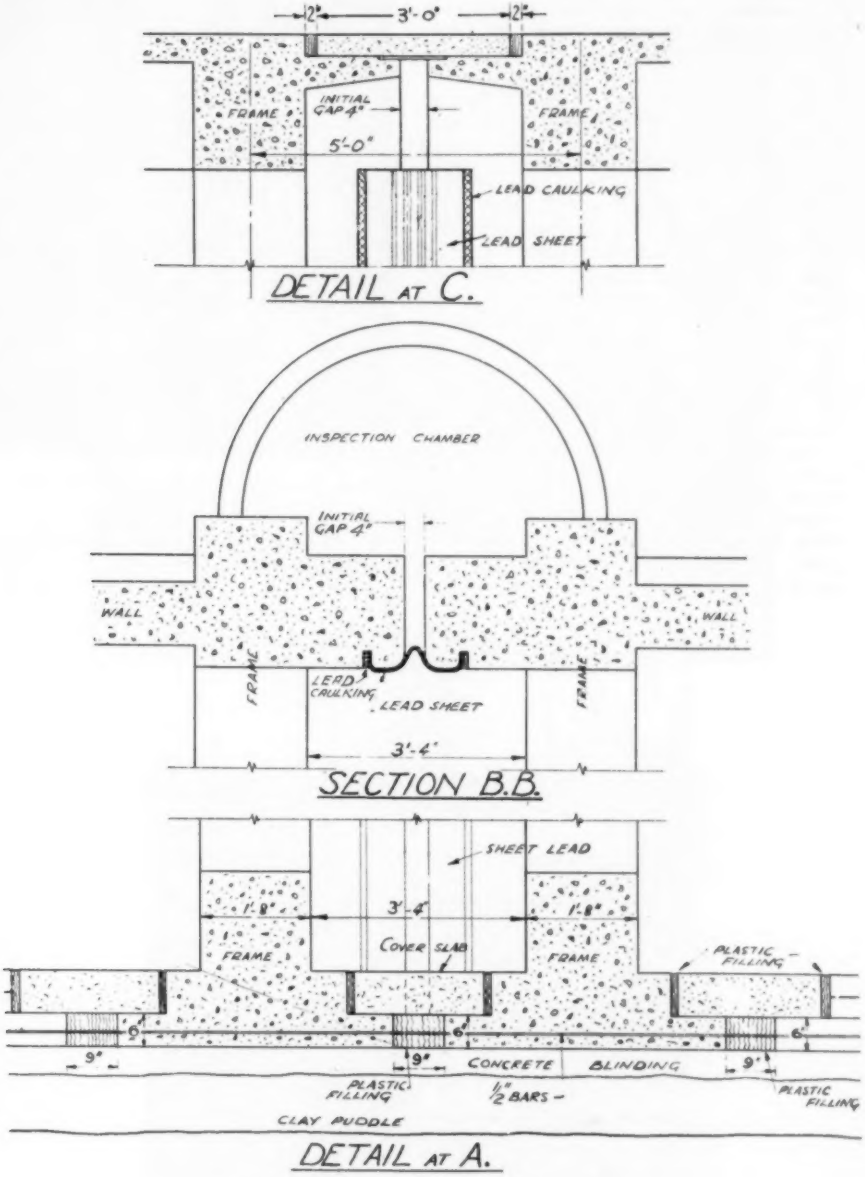


Fig. 3.—Details at Joints. (See Fig. 2.)

is capable of offering effective resistance to pressures not exceeding 3 tons per square foot. If the reservoir were founded directly upon the strong underlying ground, which would not yield under a pressure of less than 10 tons per square foot, the assumed supporting area would have to be greatly reduced and the effective spans of the units greatly increased.

The walls retaining water span from floor to roof and, in this respect, are assumed to be completely fixed at the base and freely supported at the top. They are also designed to act as deep beams of length equal to that of the units. A hinged joint similar to that between the units is provided between the foundations of the walls and girders and the floor, thereby limiting the weight of water carried by the walls and girders.

The floor is of laminated construction, formed of three thin slabs capable of following the changing contour of the ground without serious cracking. The slabs were concreted in panels, the construction joints in each of the layers being staggered. Open joints ¾ in. wide in the top slab are filled with plastic composition. The hinges (*Fig.* 4) in the floor are formed by gaps 9 in. wide between the founda-

Fig. 4.—Joints in Wall and Floor.

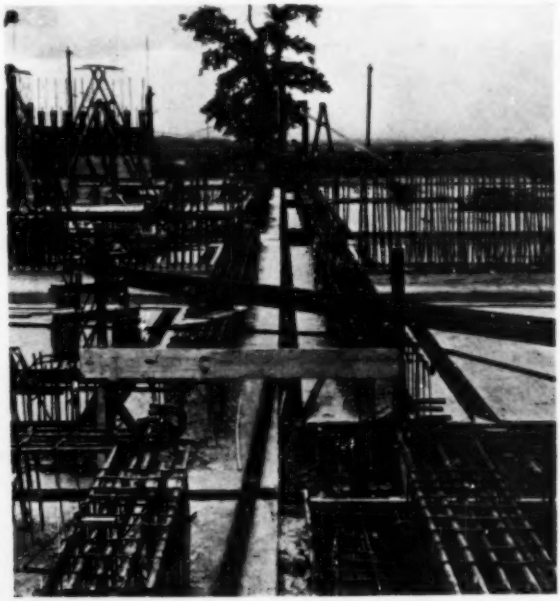


Fig. 5.—Joint in Floor and Reinforcement for Bottom of Girders.

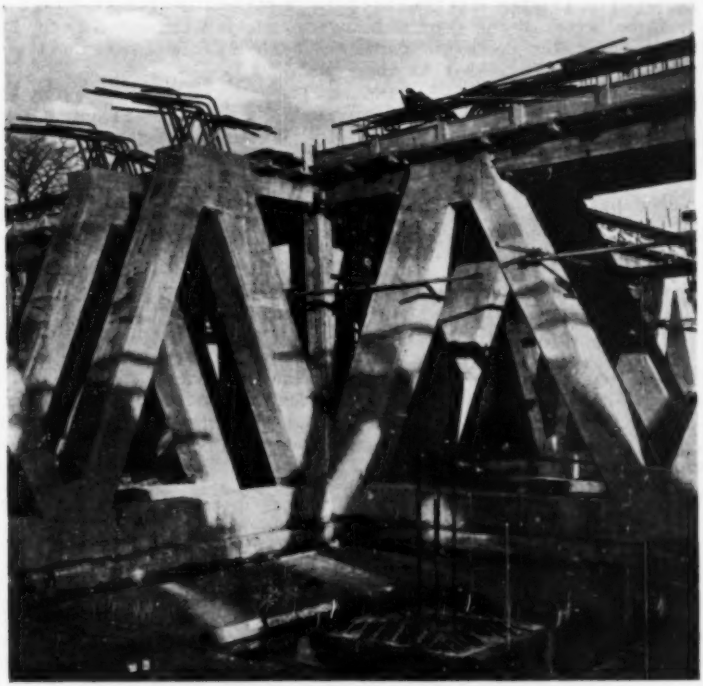


Fig. 6.—Lattice Girders and Column Foundation.

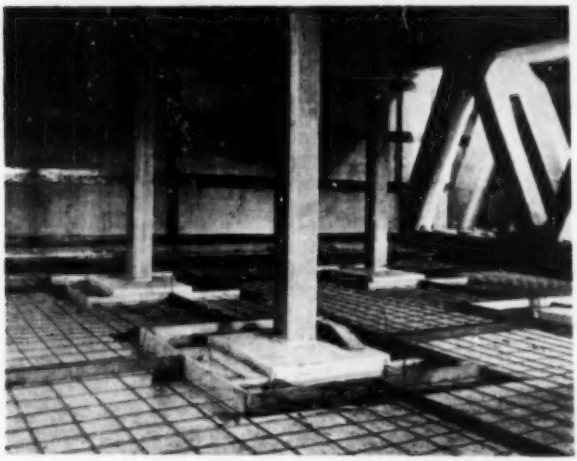


Fig. 7.—Interior Before Completion of Floor.

tions of the walls and girders and the lowest of the three layers of the floor slab. Bars of ½ in. diameter cross the gaps horizontally and are embedded in the concrete on either side at the level of the centre of the lowest layer. The gaps are filled with a plastic composition on which the two upper layers of the floor slab are placed.

Within each unit there are four columns supporting the roof (*Figs.* 6 and 7). Each column is supported on a separate base, which is connected to the floor only by flexible joints of the same type as elsewhere.

The reservoir was designed by Mr. B. W. Davies, Engineer and General Manager of the Water Department of the Corporation, in collaboration with the consulting structural engineers, Messrs. Ritchie & Partners. The contractors are Messrs. Currall, Lewis & Martin, Ltd.

Economical Construction of Cooling Towers.

It is claimed that the method of construction used for two hyperbolic cooling towers 260 ft. high recently built at Pretoria effected a considerable saving in cost compared with other methods used in South Africa. A lattice steel mast (*Fig.* 1) at the centre of the tower provided a shaft for the lift for men and materials and a support for the rotating horizontal double-cantilever arms, which could be extended or retracted to suit the varying radius of the tower. The concrete was mixed outside the tower and conveyed in a skip to the mast, up to the working level, and then along the arms to the shutters. The shutters were steel-faced timber panels 2 ft. by 4 ft. adjustable in width to conform to the perimeter of the wall. No scaffolding was required from the ground, but a scaffold provided at working level around the circumference of the tower comprised brackets of steel angles stiffened by steel tubes attached to the inner and outer faces of the wall. A gangway 4 ft. wide was formed by laying boards on the brackets. The scaffold was raised after each lift of concrete had been placed.

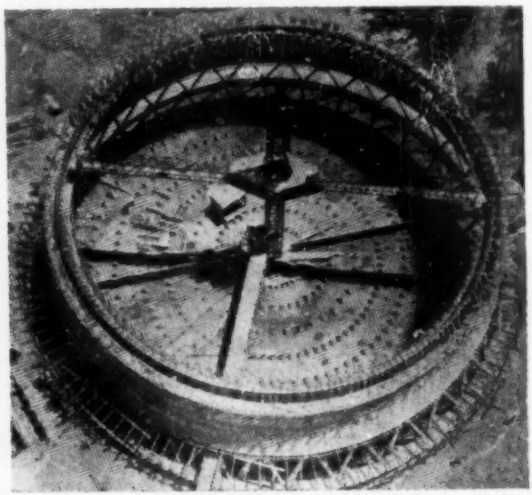


Fig. 1.

Nine-Story Office Building in London.

RAPID CONSTRUCTION.

A NEW office building in St. Giles's High Street is typical in design and method of construction of several built since the war in London. It is a reinforced concrete frame structure with brick walls. The columns and floor slabs were cast-in-situ. The beams are generally precast except those which support one or more smaller beams. Many of the stairs and landings were also precast.

There are four nine-story blocks separated by three expansion joints (*Fig.* 2). The width of each block is 48 ft. 9 in. and the height about 95 ft. There is a basement, and a retaining wall around part of the site. The foundation is generally ballast on which the bearing pressure does not exceed 2 tons per square foot. The structure is designed for the loads and stresses recommended in British standard codes of practice, the London County Council granting waivers for this purpose. The ground floor is designed for a load of 200 lb. per square foot to allow for debris. The roofs are flat except that over the area enclosed by three of the blocks there is, at the level of the first floor, a " shell " roof 72 ft. long comprising two circular-arc vaults each about 36 ft. wide. Typical details of a precast reinforced concrete beam are shown in *Fig.* 3 and a stair and landing in *Fig.* 5 ; the reinforcement in the shell roof is shown in *Fig.* 4. The precast members were made on the site and erected by one of two monotower cranes which covered almost the whole site.

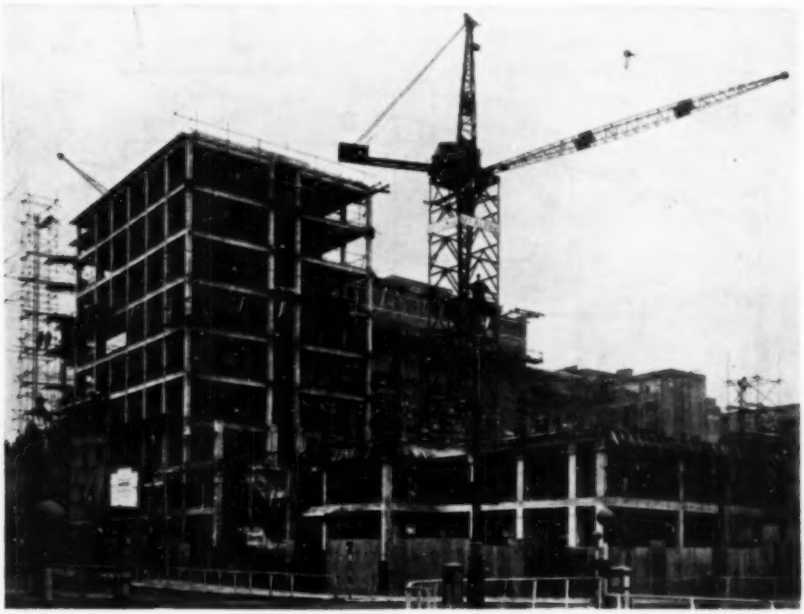
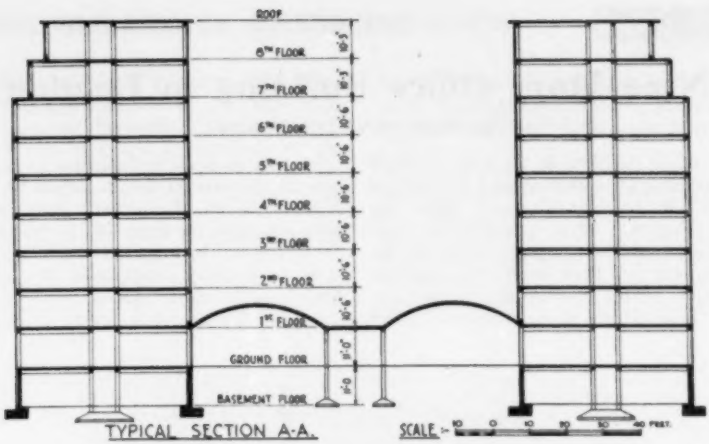


Fig. 1.—Building During Construction.

The first step in the construction of each story was to cast the columns up to just below the soffits of the beams. Wooden shuttering was used for the concealed faces of the columns, and composition boarding was used to give a smooth finish to exposed faces. The concrete was



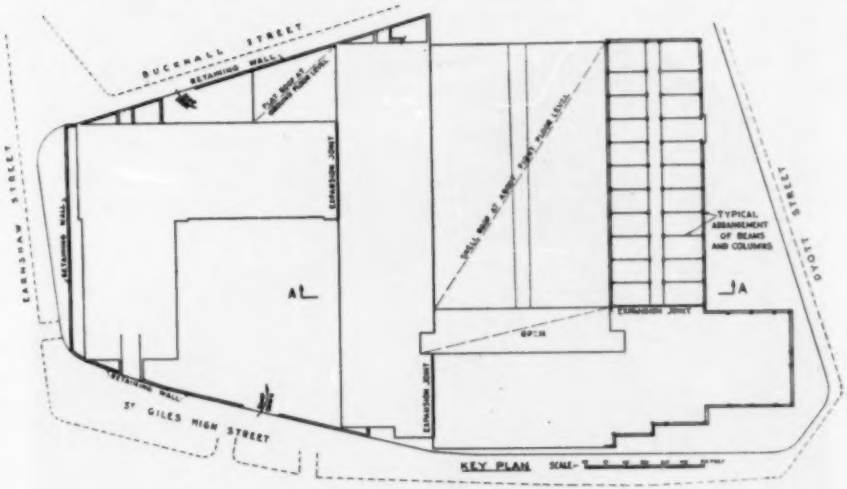


Fig. 2.—Plan and Sections.

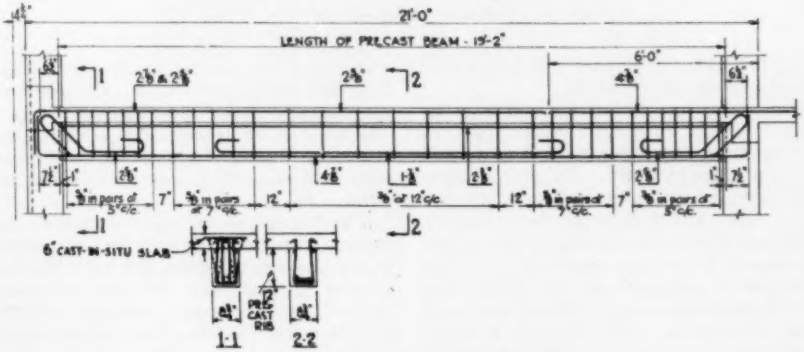


Fig. 3.—Typical Precast Beam.

brought by hoist from the mixer in ¾ cu. yd. steel skips. The skip was then suspended from the crane at four points and transferred to one of the column shutters, and remained suspended while the concrete was discharged in small quantities into an inclined trough attached to the skip and thence into the shutter. Three men were required for placing and consolidating the concrete in the columns. When the shuttering had been removed, a column-head shutter was attached to the column. This shutter, which is patented, comprises welded steel panels, placed to span between the beams to form the shuttering for the cast-in-situ floor slab. Four men, working in pairs, placed the centres for two spans at the same time, and the four men worked together to remove the centres. Reinforcement projecting from the beams ties them into the slab and junction at the column head.

The precast stairs and landings were cast (*Fig.* 6) in wooden moulds. When they were hard the castings were turned over and lifted by tackle suspended from the crane and attached to four eyes embedded in the concrete, and placed in

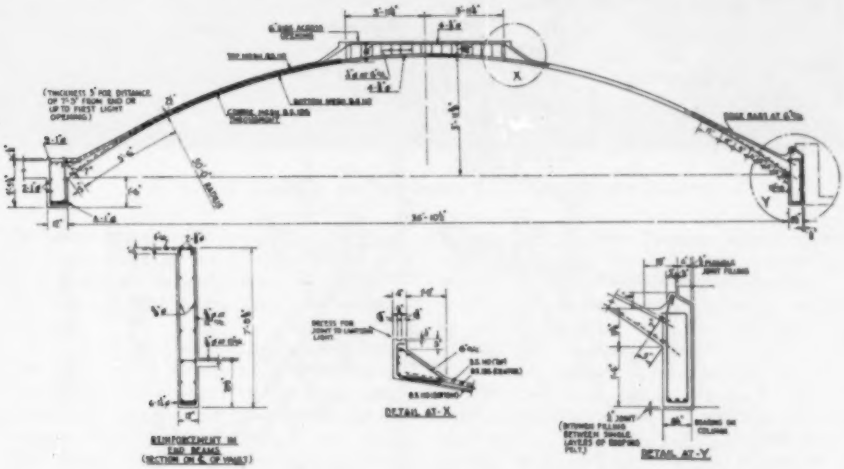


Fig. 4.—Details of Shell Roof.

24 in. and 7 in. wide and about 2 ft. deep, and one to four triangular steel brackets which provided seatings for the ends of the precast beams supported on the column. The edges of the panels and brackets were perforated with holes at close centres so that they could be assembled to suit the varying dimensions of the columns and beams at each column head. The shutters, some of which are seen in *Fig.* 6, were secured by wooden battens and bolts.

The next step was to place the precast beams in position. A beam was suspended from a two-point tackle and lowered on to the seatings. Two men guided and set the beam in position. When adjacent parallel beams had been placed, triangular steel centres were position on cast-in-situ cranked beams. Each staircase between two floors comprises three precast members, namely the lower flight and lower quarter-landing, the intermediate flight, and the upper quarter-landing and top flight.

The shuttering for the vaulted shell roof comprised light steel plates bent to the profile of the vault. Each plate was stiffened by seven triangular steel ribs. The plates were supported on steel tees also bent to the shape of the vault and supported on eight longitudinal steel tubes carried in forks attached to the head of vertical steel tubes. Shuttering was erected for the two vaults at the same time.

The steel barrows used on this site were of a type patented by the contractors.

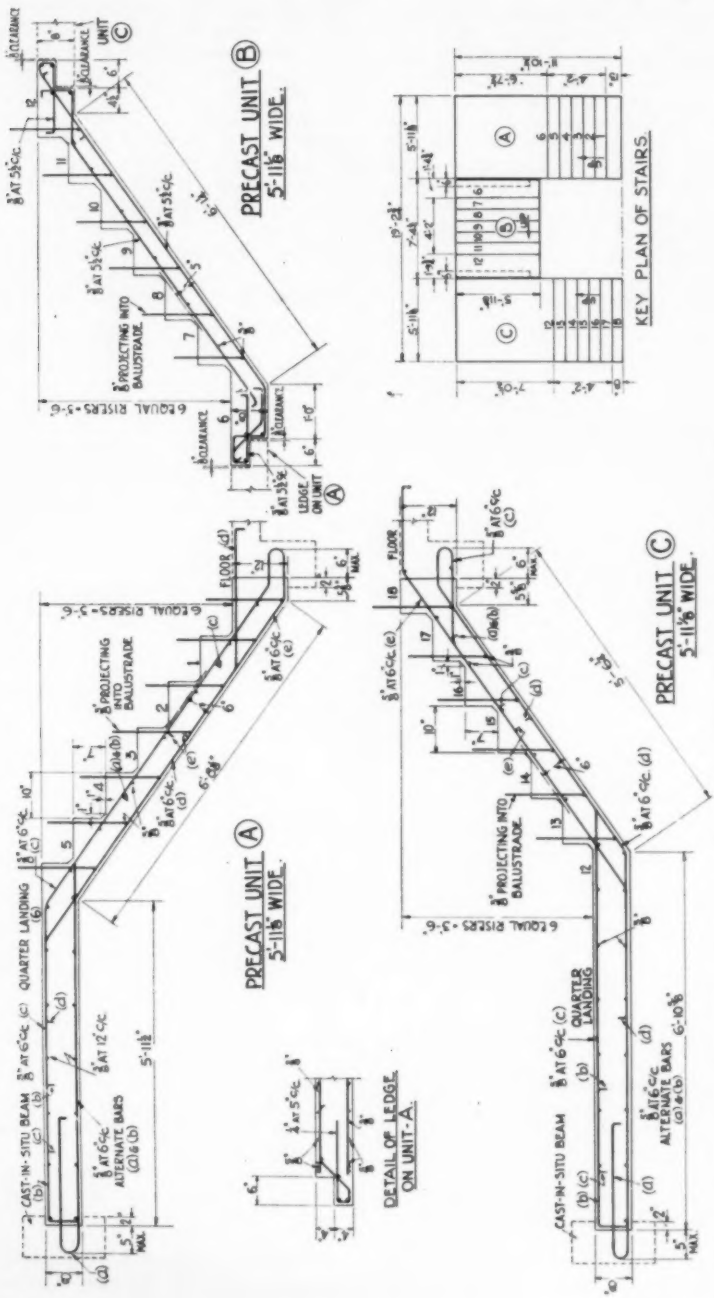


Fig. 5.—Details of Stairs and Landings.

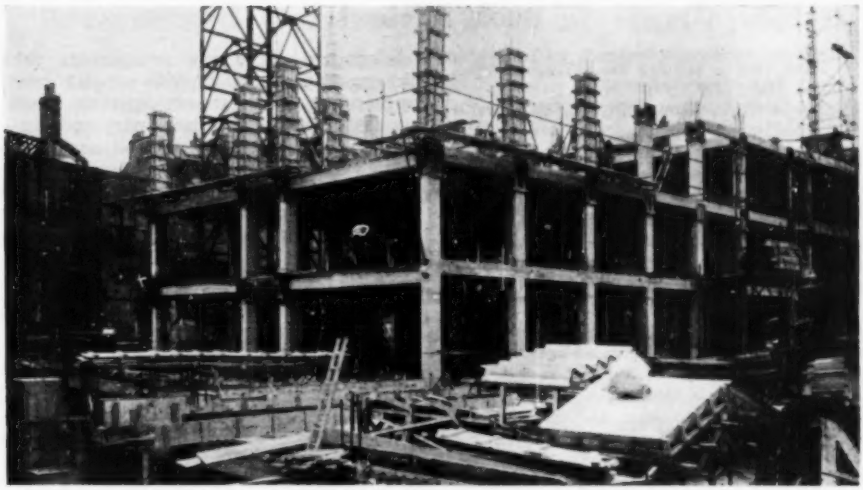


Fig. 6.—Shuttering for Columns and Moulds for Stairs.

The handles are waist high when the barrow is discharging, and the wheel is not far forward so that the force required to raise the handles is less than that usually required.

In one of the structures, nine stories were erected in seven weeks, and even during the bad weather of the winter and spring of 1950–1 progress was at the average rate of one story in ten days. [The frame and floors of a ten-story building covering an area of 53,000 sq. ft., and known as Charles House, Kensington, were erected by the same contractors by similar methods within eight months.]

The architects for the building, which is for occupation by the Ministry of Works, are Messrs. Lewis Solomon & Son. The consulting engineer is Mr. W. V. Zinn, with whom Mr. C. V. Blumfield collaborated in the design of the shell roof. The contractors for the reinforced concrete work were Sir Robert McAlpine & Sons, Ltd.

Tests of Concrete Roads.

Some data are given in the Journal of the American Concrete Institute for June, 1951, relating to 2½ miles of concrete road in Allegany County, U.S.A., laid in the year 1942 as part of a long-period study of the behaviour of concrete. The road was laid in sections in which different cements and water-cement ratios and different types of joints were used, and in some of which air-entraining chemicals were used. The observations were made 7½ years after the road was built, and relate to the condition of the concrete only. It is stated that there is no appreciable difference in the durability of concretes with slumps of 3 in. and 6 in., and that none of the many cements from different makers appeared to be superior to the others. Six different air-entraining cements were used, and in all cases these are stated to be more resistant to surface scaling than the cements without an air-entraining chemical. Where cracks have appeared, these are mostly due to weak foundations.

Book Reviews.

"Structural Theory and Design." By J. McHardy Young. Vol. II. (London: Crosby Lockwood & Son, Ltd. 1951. Price 25*s*.)

IN this book, the first volume of which was reviewed in this journal for July 1951, the author considers more advanced problems related to statically-indeterminate structures which are dealt with comprehensively and by various methods. The examples are of a general nature and are applicable in most cases to structures of steel or reinforced concrete. Detail design relates mainly to steelwork. In the consideration of soils and earth pressures, some of the terms and factors used in soils mechanics are defined and calculated, and in the chapter on foundations it is shown how these results can be used. The ordinary theories of earth pressures are also given fully. Although the book contains nearly 600 pages, it is impossible for all the subjects mentioned in the contents to be dealt with in sufficient detail to be useful to the designer. One such subject is prestressed concrete, but there are numerous references to other published works to which the reader is referred for further information or from which the author acknowledges obtaining some of the subject matter.

"Strength of Materials." By G. A. Olsen. (London; George Allen & Unwin, Ltd. 1951. Price 32*s*. 6*d*.)

THIS American book deals in about 400 pages with the elementary theory of structures as applied mainly to construction in steel and to machines. The subject is taken as far as fixed and simple continuous beams. There is no reference to concrete. A good feature is the clarity with which the principles are illustrated by sketches.

"Building and Civil Engineering Plant." By S. Geddes. (London: Crosby Lockwood & Son, Ltd. 1951. Price 30*s*.)

MUCH data on the purchase, application, costs, output, and operation of most types of machines and equipment used in civil engineering and building are brought together in this book. The data, most of which is given in tabular form, should be useful in enabling contractors to supplement their own experience. The author makes many comments of a practical nature and, although some of these may be self-evident, something is gained by their restatement. Although the illustrations and data cannot avoid being related to proprietary equipment, the danger that a book on this subject may be merely an elaborate catalogue has been avoided. Excavating machinery and concrete plant are dealt with comprehensively, but the space devoted to cranes seems meagre compared with that devoted to, say, air-compressors and pneumatic tools, crushing and screening plant, and pumps.

"Die n-freien Berechnungsweisen des einfach bewehrten, rechteckigen Stahlbetonbalkens." By K. B. Haberstock. 1951. (Berlin; Wilhelm Ernst & Sohn. Price 6 D.M).

IN this report, issued on behalf of a German committee on reinforced concrete research, the author presents methods of calculating singly-reinforced rectangular concrete beams without the use of a modular ratio. The report is in three sections. In the first the position of the neutral axis is assessed from consideration of the actual deformations and strains at the edges of the beam. In the second part strains are omitted and the theory of plasticity resorted to whereby the neutral axis is determined from conditions of equilibrium for a uniform distribution of the compressive stresses in the concrete. In the third part calculations are based on empirical assumptions which are claimed to agree with actual conditions observed at failure of a beam. The author claims that design according to the plastic theory permits a truer estimate to be made of the compressive resistance of concrete in bending which may result in the use of less reinforcement.

"Die Versuchsberichte des Deutschen Ausschusses für Stahlbeton." By O. Graf. (Berlin: Wilhelm Ernst & Sohn. 1951. Price 5.50 D.M.)

THIS is the 105th report of a German committee on reinforced concrete and summarizes the previous reports which were issued over a number of years and which are not now readily available. It is stated that there is little likelihood of further research being made for some time, and this report is therefore valuable since it gives concisely the results of research in the recent past.

"Building Technicians' Diary, 1952." (London: Association of Building Technicians. Price 5*s*.)

A POCKET-SIZE diary with maps of Great Britain, pages of graph paper, and about a hundred pages of memoranda.

Prestressed Concrete Tanks at Hartlepool.

FIVE tanks have recently been built at the Palliser Works, West Hartlepool, of the British Periclase Co., Ltd., for the Ministry of Supply. Two of these tanks have a capacity of two million gallons each, and are claimed to be the first tanks to be completed in Great Britain in prestressed concrete. The Magnel-Blaton system was employed. The tanks are for use in the production of magnesia from sea-water. Two of the tanks are for the storage of sea-water and are 106 ft. diameter and 41 ft. deep; one is for removing sand from the sea-water and is 50 ft. diameter and 46 ft. 6 in. deep; the other two are 30 ft. diameter and 36 ft. deep and in these takes place the process which produces magnesia. The stresses in the walls are lower than are generally used in prestressed concrete, because it was thought that thick walls would be desirable to permit the concrete to be easily placed and compacted, but experience has shown that thinner walls can be satisfactorily concreted. Typical cross-sections are shown in *Fig.* 1.

The vertical wires were stretched first and then the horizontal wires, so that the tensile stresses resulting from the temporary vertical bending of the walls while the horizontal wires were being stretched were counteracted by the vertical compression and no horizontal cracks could occur. The amount of pre-compression in the walls is such that with any depth of liquid they will everywhere be in compression.

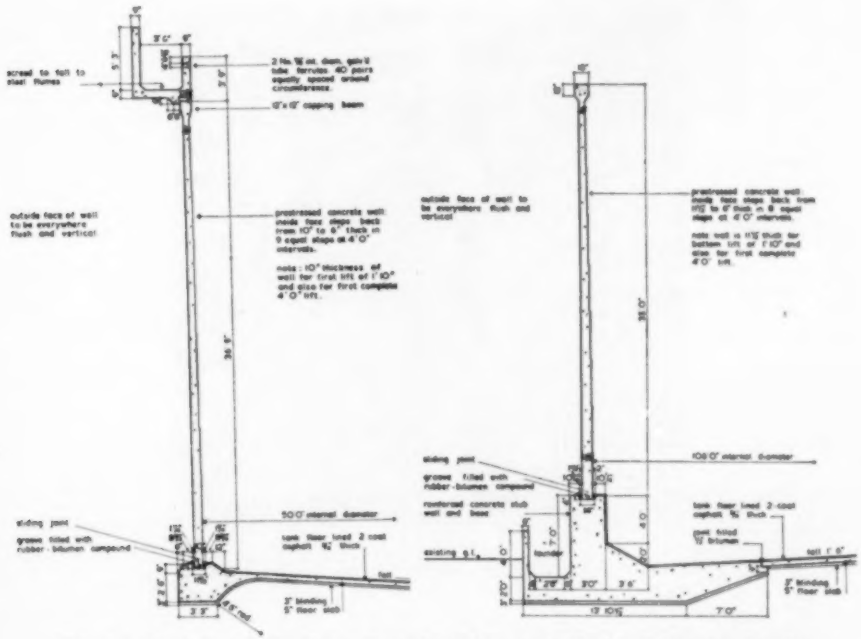

screed to fall to steel flumes
outside face of wall to be everywhere flush and vertical
prestressed concrete wall inside face steps back from 10" to 6" thick in 9 equal steps at 4'0" intervals.
note: 10" thickness of wall for first lift of 1'10" and also for first complete 4'0" lift.
sliding joint
grease filled with rubber-bitumen compound
50'0" internal diameter
tank floor lined 2 coat asphalt ½" thick
fall
3" blinding
5" floor slab
outside face of wall to be everywhere flush and vertical
prestressed concrete wall inside face steps back from 11½" to 6" thick in 8 equal steps at 4'0" intervals.
note wall is 11½" thick for bottom lift of 1'10" and also for first complete 4'0" lift.
sliding joint
grease filled with rubber-bitumen compound
reinforced concrete slab wall and base
106'0" internal diameter
tank floor lined 2 coat asphalt ½" thick
joint filled ½" bitumen
existing g.l.
fall 1'6"
3" blinding
5" floor slab

Fig. 1.—Part Cross-Sections through 50-ft. and 106-ft. Diameter Tanks.

The horizontal wires were held in position on the external surface of the wall by vertical hangers formed from 1-in. strips of 16-gauge mild steel; notches in the strips ensured the correct spacing of the wires (*Fig.* 3). After the horizontal wires had been tensioned a light steel mesh was fixed to them (*Fig.* 4) and 1 in. of mortar was applied pneumatically over

the whole outer surface of the wall as a protection to the wires and their anchors and to bond the wires to the wall.

The vertical prestressing was done with cables at the mid-thickness of the walls. For the 106-ft. diameter and the 50-ft. diameter tanks the cables were composed of sixteen 0·2-in. diameter wires and for the 30-ft. diameter tanks cables comprising eight wires were used. These cables were placed in position before concreting and the lower anchors were cast in the first lift of the wall. From 1 ft. 6 in. above the lower anchors the cables are in a duct formed by a tube through which the cables passed, and after each lift of concrete the tube was raised for the succeeding lift. At the bottom of the duct there is a hole to the outer surface of the wall through which grout was pumped to the full height of the wall until it covered the sandwich-plate anchors at the top.

Between the wall and the bottom of the tanks a sliding joint was formed by casting the wall in a groove in the bottom.

In the largest tanks openings for pipes up to 3 ft. 3 in. diameter were formed near the bottom of the wall. Therefore the lower 6 ft. of height of the wall and the launder around the tanks to collect overflow are in reinforced concrete, and there are no holes in the lower part of the prestressed wall. The floor, which has a fall of 2 ft. from the centre to the wall, is also in reinforced concrete and is 5 in. thick on a 3-in. screed. The floor and inner surface of the reinforced concrete part of the wall are finished with ¾ in. of asphalt placed in three layers. At the top of the reinforced concrete part of the wall, which is 3 ft. thick, a groove 4 in. deep was formed in which the prestressed part of the wall rests. The lower surface of the groove was finished with a steel trowel and then given a coat of hot bitumen in order to form a sliding joint between the prestressed and reinforced concrete parts of the wall. The space between the prestressed part of the wall and the sides of the groove in which it is built is sealed with rubber-impregnated bitumen on the inner side ; on the outside of the wall the space is also filled to prevent blown sand from entering it and restricting the movement of the prestressed wall. The prestressed wall is therefore 35 ft. high, and it is reduced in thickness from 11½ in. at the bottom to 6 in. at the top by steps

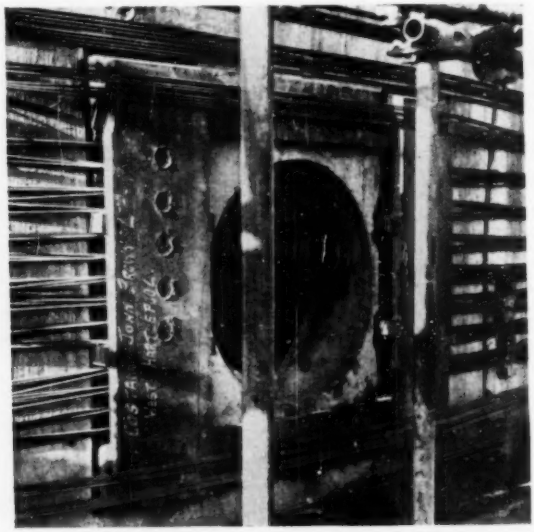


Fig. 2.—Steel Frame Around Manhole.

on the inside ; the outer surface is vertical throughout. The inner surface of the prestressed wall has no protective coating.

The 50-ft. diameter tank, which is used for removing sand from the sea-water, also has a reinforced concrete floor in the centre of which is a drain for the removal of the sand. The prestressed wall rests in a groove in the floor, and varies in thickness from 10 in. at the bottom to 6 in. at the top, being stepped on the inner face. Near the bottom of the wall there is a manhole 2 ft. 6 in. diameter, on either side of which the vertical cables pass (*Fig.* 2). The pull in the horizontal wires is carried around the opening by a steel frame to which the wires are anchored. This frame was erected after concreting had been completed, and is held in place by the wires which are anchored to it or which pass in front of it at the top and bottom. At about mid-height of the wall two holes were formed, each to accommodate two 18-in. diameter pipes. A steel frame enclosing all four pipes transmits the horizontal prestressing force around the holes, and the vertical cables pass between or outside the holes. On the top of the tank there is an annular launder, 3 ft. 6 in. wide inside and with an outer wall 6 ft. high and an inner wall 3 ft. 6 in. high, which forms the weir over which the water discharges from the sand trap. The launder is in reinforced concrete and there is a copper water-bar

Fig. 3.—Supports for Horizontal Wires.

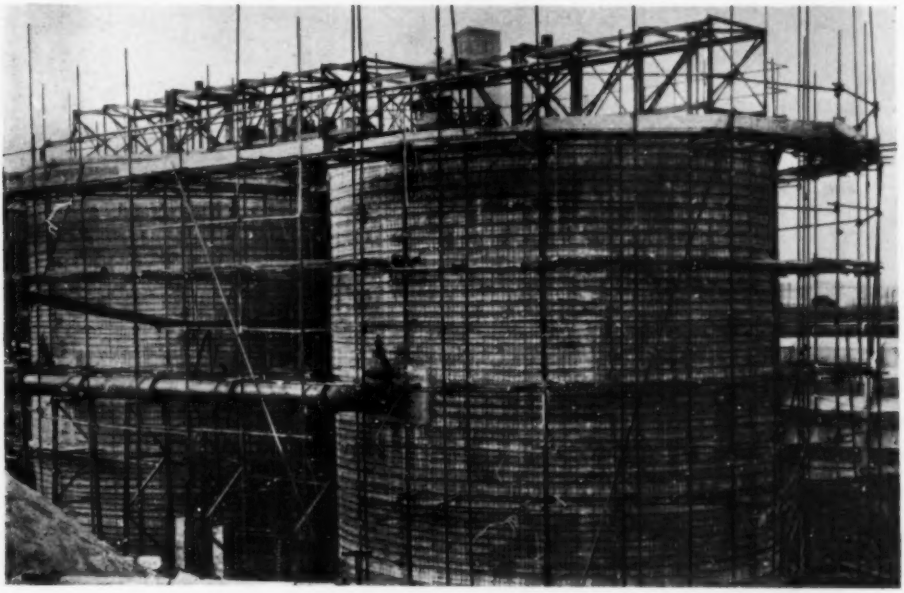


Fig. 4.—Prestressing Completed and Mesh in Position for Gunite.

between the top of the wall and the launder.

The 30-ft. diameter tanks are similar to the 50-ft. diameter tank except that the wall varies in thickness from 9 in. to 6 in.

Vertical Wires.

The bottom anchors for the vertical wires comprise a short mild steel bar to which are welded several ¼-in. mild steel links cast in the concrete of the first lift (*Fig.* 5). Each wire in the cable passes from the top anchor, around this bottom anchor, and back to the top. The top anchors (*Fig.* 6) are standard sandwich-plates and distribution plates. The cables were stretched by Magnel-Blaton jacks (*Fig.* 7). The cables were made in the usual manner except that they have only one grille near each end to keep the wires in their correct positions. The 0·2-in. diameter wires have a tensile strength of 100 to 110 tons per square inch and the stretching force applied was 140,000 lb. per square inch.

Horizontal Wires.

The anchors for the horizontal wires are the opposed-type sandwich-plates which have four grooves arranged so that when the ends of eight wires are secured the plate lies flat against the concrete. The

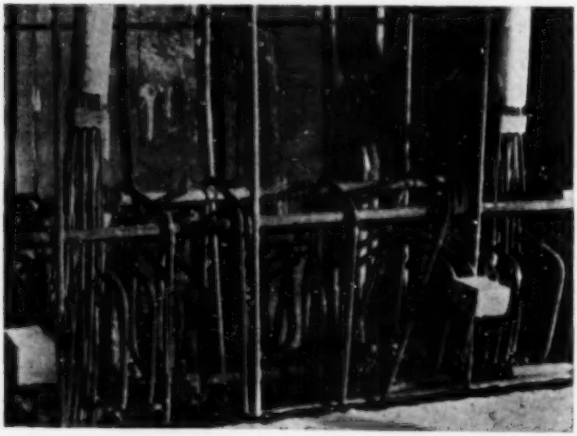


Fig. 5.—Reinforcement and Anchors of Vertical Wires in Bottom of Wall.

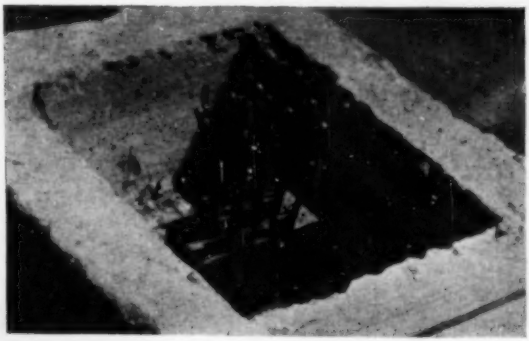


Fig. 6.—Top of Vertical Wires after Stretching.

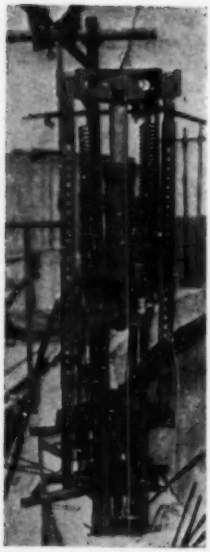


Fig. 7.—Jack for Stretching Vertical Wires.

wires were stretched by a jack (*Fig.* 8) which straddles the sandwich-plate and pulls two wires from each direction. When the required extension was obtained the wedges were secured in the grooves of the sandwich-plate. The wires in the upper part of the wall are 0·2 in. diameter and have a tensile strength of 100 to 110 tons per square inch. The wires in the lower part of the wall are 0·276 in. diameter and have a tensile strength of 95 to 105 tons per square inch.

simultaneously at four points 90 deg. apart. In the smaller tanks the wires were tensioned at two points 180 deg. apart. The anchors of the wires in adjacent groups are spaced around the wall to provide a uniform prestress. The horizontal wires are in groups of four, the wires in each group being lightly clipped together at intervals of about 10 ft. before being placed in position on the hangers which had been previously fixed to the walls.

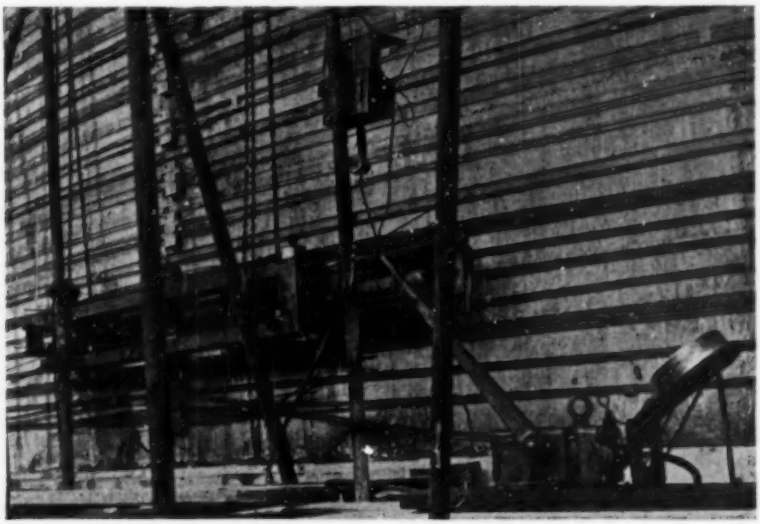


Fig. 8.—Jack for Stretching Horizontal Wires.

The friction between the wires and the concrete caused the stress in each wire to vary from a maximum at the jack to a minimum at the point farthest from it. This variation was controlled by tensioning each pair of wires simultaneously at several points around the tank, with an anchor at each tensioning point. The number of tensioning points to give the most economical result depends upon the relative cost of labour, wire, and anchors. In these tanks it was required that there should be only one band of horizontal wires at any point on the tank. In the most highly stressed part near the bottom of the 106-ft. diameter tanks the wires were tensioned and anchored

Construction.

The concrete was a 1 : 1½ : 3 mixture with a water-cement ratio of 0·45. Shutter vibrators were used on the prestressed walls and launders and immersion vibrators for the reinforced concrete part of the walls and bases. Ordinary Portland cement was used, and 28-day test cubes had an average compressive strength of 6500 lb. per square inch. The floors and the reinforced concrete parts of the walls of the largest tanks were concreted in alternate bays in order to reduce shrinkage. The prestressed walls were cast in lifts of 4 ft., each lift being placed in a continuous operation and completed

before the next lift was commenced. The surface of construction joints between lifts was cleaned by removing the scum and laitance from the concrete a few hours after casting by means of a water spray. This surface was kept damp until just before the next lift was placed, when a layer of mortar was spread on it and tamped. During the prestressing operation three gauges were placed between the bottom of the prestressed wall and the reinforced concrete base, and these showed reasonable correspondence with the calculated contraction under prestressing. Readings taken when the tanks were being emptied and filled showed that the prestressed wall moved freely in the joint.

The chief engineer of the Periclase Co., Ltd., is Mr. P. J. Metcalf. The contractors were Messrs. Costain–John Brown, Ltd. The prestressing equipment was supplied by Stressed Concrete Design, Ltd.

Deflection of Reinforced Concrete Beams.

As a result of tests made at the Universities of Tasmania and Cambridge by Mr. F. A. Blakey, and described in a recent number of "Magazine of Concrete Research," the author states that the deflection caused by short-term loading of a freely-supported concrete beam reinforced in tension only may be calculated from the equations

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \quad . \quad . \quad . \quad . \quad (1)$$

$$\frac{d^2y}{dx^2} = \frac{M}{E_s p b d^3 (1 - n_1)\left(1 - \frac{n_1}{3}\right)} \quad (2)$$

Equation (1) is valid up to the limit imposed by the modulus of rupture of the concrete; E is the secant modulus of elasticity, and I is the moment of inertia of the transformed section. In equation (2), E_s is the elastic modulus of steel, and n_1 (the neutral-axis factor) is calculated on the assumption of linear stress distribution in the concrete; b and d are the breadth and effective depth of the beam; p is the proportion of tensile reinforcement. This equation, it is stated, appears to be valid up to the load at which the reinforcement yields.

The author states that the accuracy of the method depends on the accuracy with which the modulus of rupture of the concrete can be determined, and hence the upper limit of the validity of equation (1), and also the accuracy with which n_1 may be determined when the modular ratio is obtained from an empirical expression. The combined use of equations (1) and (2) is superior to the use of equation (1) alone for the calculation of the deflection of most of the beams tested at any load up to the ultimate load. This method has the advantage that it starts from the same assumptions as the ordinary procedure for calculating the stresses in a reinforced concrete beam. The agreement is not so satisfactory for beams with a large percentage of reinforcement.

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Correspondence.

THE THEORY OF PLASTIC HINGES.

From DR. R. GARTNER, D.Sc.(Eng.).

I read with great interest the Editorial Note on the theory of plastic hinges in your number for September 1951. May I be permitted to add the following?

In the "Publications" of the International Association for Bridge and Structural Engineering for 1933–4 appeared an article on "The Theory of Plasticity in the Investigation of Statically-indeterminate Supporting Structures of Reinforced Concrete", by Dr. A. Frendenthal, of Prague, in which the theory of plastic hinges is expounded. There are still earlier publications dealing with this subject, so that it is at least two decades since the theory was propagated.

Without confusing it with the plastic or ultimate-load theory of reinforced concrete beams, the effect of the plastic-hinge theory is to give a comparatively wide choice of the moment-diagram for an indeterminate structure, whereas one moment-diagram only applies in the elastic theory.

The advantages of the plastic-hinge theory are therefore: (1) The distribution of moments can be arranged according to the wishes of the designer, so that, for example, massing of reinforcement over supports can be avoided. (2) On account of the possible variation of the moment-diagram, the solution of the equations for the unknowns of the indeterminate structure need not be exact; the unknowns can be chosen freely between two limits, making their calculation much easier.

In reinforced concrete construction there are three different possibilities of developing plastic hinges, namely, (a) by exceeding the yield point of the steel, (b) by exceeding a certain value of the ratio of the elastic to the non-elastic deformation of the concrete, and (c) when cases (a) and (b) occur together. The possibility in case (a) is uncontested. In regard to case (b), many results of tests confirm that non-elastic movements can replace the plastic behaviour of the steel; see, for example, Building Research Station Bulletin, "Moment Re-distribution in Reinforced Concrete" (1939), by Glanville and Thomas.

As you mention that one should not be satisfied with a few tests only, may I point out the following? The British Standard Code of Practice allows negative moments to be reduced by 15 per cent. and positive moments to be increased accordingly on account of the ultimate load-carrying capacity, etc. The German regulations stipulate that, for concrete with a compressive strength of 2400 lb. per square inch, the allowable stress of 750 lb. per square inch for beams can be increased over the supports to 1050 lb. per square inch. This recommendation does not seem to be a result of the plastic theory, but, as the quality of the concrete in the beam will be about the same throughout, it amounts to a decrease of the support moment of 30 per cent. allowing for the greater amount of steel. The fact that structures designed in accordance with this recommendation have been erected for many years without any sign of undue deformation or other distress is surely as good as a large number of tests.

The lower limit of plastic-hinge moments "with safety included" can therefore be assumed to be at least 70 per cent. of the elastic moment; that is a moment of $\frac{WL^2}{10}$ can be reduced to $\frac{WL^2}{14}$. I think that even this figure is conservative, but at least a beginning could be made with it and the adjustment of 15 per cent. permitted in the B.S. Code could safely be increased to 30 per cent. for all indeterminate structures.

A complete analysis cannot be given here, but the following may be mentioned. Any n-times statically-indeterminate structure (or part of a structure) develops at the ultimate load $n + 1$ hinges, so that it becomes a mechanism. The structure can always be designed in such a way that, apart from the last plastic hinge which makes the structure an unstable mechanism, all the other plastic hinges develop over supports or at the corners of frames. These latter plastic hinges may (for the solution of the unknowns) be taken as real hinges to make the system determinate. In other words, the moments at the hinge points are the unknowns. Between these hinge points the members form free beams. The unknowns are the partial fixing moments; they can be chosen freely between two limits and form

the base-line of the moment-diagram. As the stresses are almost proportional to the loads, the design may be based on the given loads and ordinary permissible stresses, instead of ultimate loads (which are the given loads multiplied by a load factor) and ultimate stresses.

It is emphasized that temperature stresses, shrinkage, etc., must be included in the calculation just as other stresses, for example stresses produced by and during erection, so far as they are not secondary stresses which are allowed for by the factor of safety.

In conclusion, in the mild form suggested here the plastic-hinge theory could be applied with confidence now and be expanded and developed as soon as further research gives more exact indications of its applicability.

THE ULTIMATE RESISTANCE OF PRESTRESSED CONCRETE BEAMS.

From MR. J. W. H. KING.

As Dr. Abeles, in the last paragraph of his article in your number for October, 1951, criticised my method of design as published in your journal for September, 1950, I trust you will allow me space to reply.

The criticism declares that my method is based on the average elongation of steel measured over one or more cracks, and that this is unsound. In the first place I would indicate that at no time did I equate strains over any finite length. This is in accordance with accepted bending theories, including those used by Dr. Abeles both for cracked and non-cracked sections in his numerous writings. In such cases the purist might say that one cannot have strain without original length, and so an infinitely small length is actually considered in evolving some of the basic theories in text-books.

As to the matter of steel stress in a crack being higher than in immediately adjacent steel, I would ask Dr. Abeles to consider two facts. First, a non-existent length of steel cannot suddenly extend to cross a crack, however small, without having an infinite stress whilst still being whole, or by rupture. It follows therefore that bond between concrete and steel adjacent to a crack must fail in order to allow a finite length of steel, however small, to stretch, and that, secondly, fundamental conceptions of statics then make it necessary for the stress over that length to be constant in the steel and, for very small lengths, in the concrete also. Hence strain distribution in the compression concrete, and in the tension steel for any section in this possibly very small length, must be materially identical, if no significant outside forces have been at work. Hence my method is still sound if Dr. Abeles' comment about average extension is considered to be valid, and a small finite length is considered. Personally I feel the point does not arise.

As a matter of fact the only case for which I made use of average extension was for non-bonded beams, for which I gave the broad outline of a method of attack. The problems of bonded and non-bonded beams are essentially different in that respect.

From DR. P. W. ABELES.

Mr. King's letter is a welcome opportunity of correcting Note II of my article. My intention was to point out that various methods of design which have been evolved, including those of Professor A. L. L. Baker and Mr. J. W. H. King, which are based on the ultimate strain of the concrete in compression and of the steel in tension, are all based on strain measurement, and that strain measurement is not peculiar to Mr. King's method alone. The words "The latter method" were intended to read "The latter strain" and to refer to the ultimate strain of the steel in tension, and not to Mr. King's method, as all these methods are based on an average elongation measured over one or more cracks.

With regard to Mr. King's assumption that the steel stress over a finite length in a narrow crack between adjacent bonded portions is necessarily constant, I may say that nobody yet has been able to measure the actual strain in a crack. The stress in the steel obviously will vary between two cracks if the bond is not destroyed. A stress redistribution is most likely to take place with concentrations similar to those in a narrow notch. This accounts for a considerable increase in the nominal ultimate strength of thin bars embedded in concrete compared with that of bars in the air. This discrepancy is the justification for my statement that all the methods employing strain measurements are also approximations, and as such are not very simple.

A Dolphin with Prestressed Concrete Piles.

THE dolphin illustrated in *Fig.* 1 is supported by five 12-in. prestressed concrete piles 40 ft. long. Four of the piles project beside, but clear of, the dolphin and are rigidly connected thereto at two levels. The other pile, which is in the centre of the group, supports the bottom of the dolphin. Resilient pads or buffers are interposed between each pile and the dolphin, and the rigid connections are above and below the point at which blows from vessels are ordinarily received. When a ship strikes one or more of the piles they deflect towards the dolphin, the elasticity of the prestressed concrete pile being sufficient to permit this deformation to take place without damage, and thus some of the impact is absorbed. When the deflection of the pile is sufficient completely to compress the buffer or buffers the resistance to further movement of the ship increases, since the resistance of the deflected pile is then augmented by that of the dolphin and the other piles.

The dolphin may have any number of piles ; if there are five, as in *Fig.* 1, four of them are at the corners of a 6 ft. square.

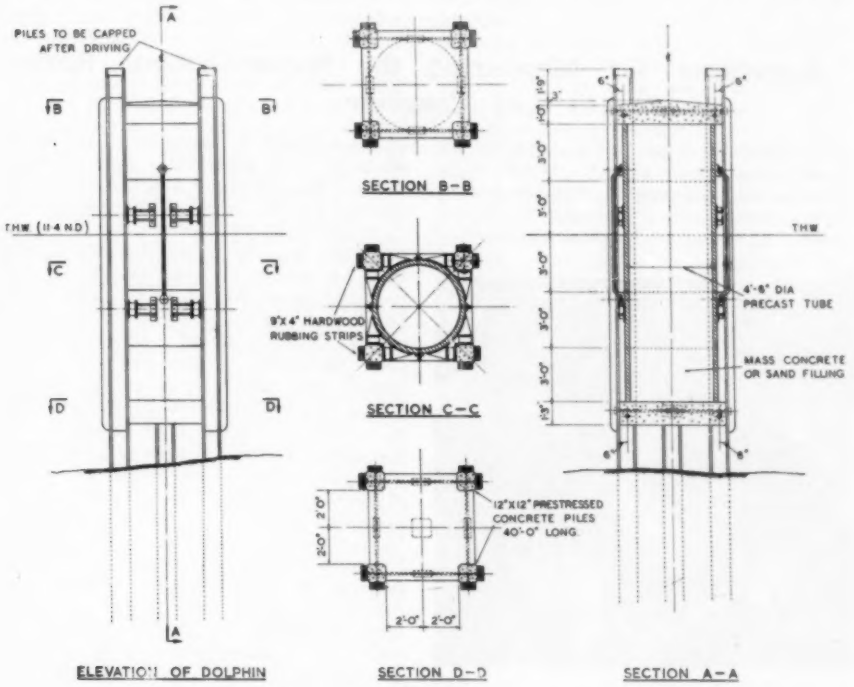

Fig. 1.

The piles have 9-in. by 4-in. hardwood rubbing strips on their exposed faces. The strip may be against the pile, or it may be clear of the pile by being attached to packing pieces ; the latter arrangement improves the shock-absorbing properties of the dolphin. At each level at which the piles are connected to the dolphin a cast-in-situ concrete slab acts as a spacer to which the piles are fixed by drawbolts.

The dolphin is constructed with

4-ft. 6-in. diameter precast concrete pipes erected vertically on the lower slab, filled with concrete or sand, and capped with the upper slab. Buffers of rubber pads between metal plates are interposed between the pile and the pipes. Alternatively spring-loaded buffers may be provided. One or more steel mooring rods may be attached to the dolphin, thereby avoiding damage due to rubbing of mooring chains or hawsers attached to the piles.

In another design of the dolphin, eight piles spaced in the form of a regular octagon are provided. With this arrangement it is unsatisfactory to connect the piles to one another by draw-bolts, so the draw-bolts are arranged radially, one end of each bolt being in the pile and the other secured to a metal ring at the centre of the group of piles. The bolts and ring are embedded in the concrete slab.

This design is patented by the North Thames Gas Board and Mr. S. V. Gardner.

Apparatus for Measuring the Water-Cement Ratio of Concrete.

THE apparatus shown in *Fig.* 1 enables the water-cement ratio of wet concrete to be ascertained or compared with the known water-cement ratio of a sample of wet concrete. It comprises a cylindrical canister

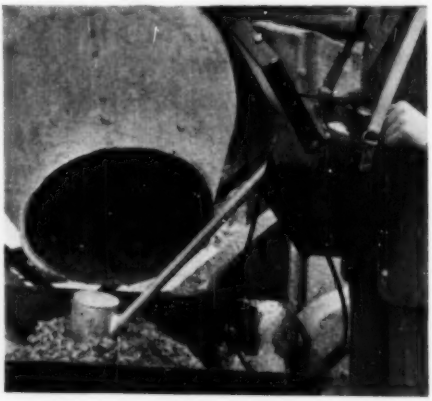

Fig. 1.

on the end of a handle and an electrical recording instrument. The canister contains a vibrator operated by electricity from batteries in the instrument box or from the mains. The electrical resistance of the grout brought by the vibratory action to the surface of the concrete between two electrodes on the underside of the canister is a measure of the water content. The higher the water content the less the resistance and the higher the reading on the gauge of the instrument.

To ascertain whether concrete as made on a site has the required water-cement ratio, a batch of concrete having the required water-cement ratio is first made and tested and the position of the pointer on the gauge is noted. The canister is then immersed in the concrete to be tested, and the position of the pointer indicates whether the water-cement ratio is higher or lower than that of the specially-prepared concrete. To determine the actual water-cement ratio of a sample of concrete, this is tested and the number against the pointer is noted. With the same materials as are used for the sample, two or three batches of concrete each with different water-cement ratios are made and tested and the number against the pointer is noted in each case. From these numbers a curve is drawn to give the relation between the numbers and the water-cement ratio, and the ratio corresponding to the number of the sample is determined by reference to the curve.

Tests at the Building Research Station showed that a difference of 0·05 in the water-cement ratio is indicated by the apparatus, which was designed and is supplied by Civil Engineering Research Laboratories, Sunbury, Middlesex.

Design and Construction of Foundations.

SEVERAL papers on the study of soils and the design of foundations were read at the Building Research Congress held in London recently. Some abstracts are given in the following.

Compaction of Soil.

The application of soil compaction to deep foundations in loose sands and soft clays is described by Mr. H. J. B. Harding and Mr. R. Glossop in " The Influence of Modern Soil Studies on the Construction of Foundations." Fine-grained soil of high porosity, such as loose sand, may cause serious settlement of buildings if the stratum is of great depth. Loose sands may become momentarily unstable if their structure is destroyed by vibration but may be stabilised by deep compaction. Pile-driving is the most common method of compaction which is effected by displacement and vibration. Another method is " vibroflotation," by which the sand is compacted in depth by vibration by an electrically-operated tool, which is jetted into the ground. The tool comprises a cylindrical housing, 3 m. long and 40 cm. in diameter, in which a 15-h.p. motor drives unbalanced weights on a vertical shaft. The vibrations have a frequency of 3000 per minute and an amplitude of 10 mm. Jetting nozzles are fitted at both ends. The vibrator is jetted down to the required depth at a rate of about 2 m. per minute, the upper jet is then opened, and the motor started. The progress of compaction is shown on an ammeter. The vibrator is withdrawn in stages. When a crater appears at the surface, sand is added. Penetrations are made at about 1·5-m. centres.

Loose sand below ground-water level may also be compacted by the explosion of buried charges. The charges, each of about 8 lb. of gelignite, are jetted down to a depth of 15 ft. at about 5-ft. centres. For great depths the charges are placed in stages. Since water and gas escape at the surface, any impermeable surface stratum must be removed before blasting. For soils above ground-water level, compaction by piling only can be used. For saturated soils, either vibration or explosions may be used.

Accelerated consolidation by sand drains was developed for stabilising embankments across a great depth of soft clay and peat. The method can be used for a structure, such as a large tank, where the cost of a piled foundation would be excessive but where long-continued settlements would be troublesome. The consolidation of the clay is accompanied by an increase of the shearing strength, and hence of the ultimate bearing capacity. The rate of consolidation of clay under load depends upon the permeability and on the length of the drainage paths whereby the water migrates; the paths can be much shortened by boreholes filled with sand. In this way, settlements which would normally continue for years are complete in a few months. Typically, the drains are 16 in. diameter and spaced about 10 ft. apart. The sand-filling communicates at the surface with a permeable layer at the base of the embankment, through which the water escapes. The boreholes are formed by ordinary methods, by jetting, or by a driven mandrel.

Where an embankment is on a site underlain by a great depth of peat which is extremely compressible, it may be necessary to remove the peat and replace it by filling. Since excavation is costly, a method of displacement by blasting has been developed in Germany, where it has been used on peat beds up to 72 ft. deep. The charges, each of about 50 lb. of gelignite, are jetted into place before the bank is built. The filling is then placed, without compaction, to a height about equal to the depth of peat to be displaced. The outer charges are fired first. The effect of the explosion is to lift the whole embankment, which then falls back on to the peat, the structure of which has been destroyed by the explosion, and displaces it sideways. Any remaining peat consolidates rapidly under the weight of the embankment.

Domed Foundations.

Mr. Harding and Mr. Glossop also describe a foundation structure, called the Durley dome, which is suitable for tanks and other structures of no great value constructed on a great depth of soft soil and when the cost of a raft or piled foundation would be out of proportion to the value of the structure. A Durley

dome (*Fig.* 1) is a concrete dome with a large amount of peripheral reinforcement. The stresses in the dome are all compressive, and no bending moments occur. The area of the foundation can be increased without undue increase in the weight of the foundation structure. No shuttering is required, since the ground is excavated to the required shape for the underside of the dome.

Soil Mechanics.

In " Building Foundations in Theory and Practice," Mr. K. Terzaghi states that the time has gone when contributions to the study of soils of great practical importance could still be made by pure reasoning or by small-scale laboratory tests, but the gap between the theory of soil mechanics and the practice of foundation engineering still widens. Mathematical investigations are still being performed on the influence of elastic anisotropy on the stress distribution in the subsoil under footings, but the error due to neglecting this influence can hardly exceed 30 per cent. On the other hand, some recently constructed buildings have to be underpinned because of excessive settlement, which may be at least ten times greater than the designer expected. The relation between the penetration of a pile and the ultimate bearing capacity of the pile is now well developed, but on the site correlation between the boring and the pile-penetration records are few.

If the soil conditions are simple, an elementary knowledge of soils combined with the most primitive methods of dry-sample borings is sufficient to prevent flagrantly erroneous forecasts. Unfortunately simple soil conditions are uncommon. If soil conditions are complex, the advanced state of soil mechanics is of no avail unless the engineer in charge of the soil exploration is aware of the virtues and deficiencies of the different methods, and capable of adapting these methods to the actual conditions. Methods of subsoil exploration cannot be standardised, and still leave a wide margin for improvement so far as expediency and reliability are concerned. Hence the development of these methods and the comparison between forecast and actual performance of foundations constitute one of the most important subjects of research in civil engineering.

Penetration Tests for Piles.

A method of determining the length and bearing capacity of piles has been devised in Holland and is described by Mr. T. K. Huizinga in " Application of Results of Deep Penetration Tests to Foundation Piles." The object of the test is to determine reliably the bearing capacity of the ground at various depths. The apparatus comprises a rod, at the lower end of which is a cone, and around which is a tube of the same diameter as the cone. The area at the base of the cone is 10 sq. cm. The tube and rod can be pressed separately into the soil. By pressing down the tube also the cone is pushed into the soil. A device at the top enables the resistance of the cone and the frictional resistance along the tube to be measured. The greatest load which can be placed on the rod or on the tube and the depth to which the ground can be investigated depend upon the characteristics of the soil and the construction of the apparatus. For the ordinary apparatus the maximum load is about 10 tons, and depths of 20 m. to 25 m. can be easily reached in Dutch soils, which are sufficient for piles of common lengths. If it is necessary to know the characteristics of the soil at greater depths the apparatus is placed in a boring tube, so that the friction along the upper part of the penetration tube is removed and the device can be forced down much deeper. A depth of 60 m. has been reached.

Fig. 1.—A Domed Foundation.

Bearing Capacity of Clay.

In " The Bearing Capacity of Clays " Mr. A. W. Skempton explains that saturated clays (and most clays are saturated) behave as if they were purely cohesive non-frictional materials if no change of water content takes place under the applied stresses; that is, the angle of shearing resistance ϕ is equal to zero. This assumption is the basis of ordinary calculations of the bearing capacity of clays. Only in special cases of prolonged loading or very silty clays is the assumption sufficiently far from the truth to justify a more elaborate analysis. In course of time, however, consolidation becomes important, and leads to long-continued settlements.

The allowable foundation pressure q may be expressed by

$$\frac{1}{F}\left[cN_c + p_0(N_q - 1) + \frac{wB}{2}N_w\right] + p$$

where F is the factor of safety, c the apparent cohesion of the soil, p_0 the effective overburden pressure at foundation level, p the total overburden pressure at foundation level, w the density of soil beneath the foundation and the submerged density if the foundation is below water level, B the breadth of the foundation, and N_c, N_q, N_w are factors depending upon the angle of shearing resistance of the soil, the ratio of length L to breadth B of the foundation, and the ratio of the depth D to B. If $\phi = 0$, $N_q = 1$ and $N_w = 0$. Therefore $q = \frac{c}{F}N_c + p$, and the bearing capacity is $cN_c + p$.

The apparent cohesion (generally referred to as the shearing strength) of the clay is determined by testing undisturbed samples from boreholes. The factor N_c has been evaluated for the particular values of B, L and D, and is given in *Fig.* 2. So far as the present evidence is concerned, the values of N_c given in *Fig.* 2 are sufficiently accurate for the determination of the bearing capacity of deep beds of relatively homogeneous clay. A factor of safety of at least 3 is desirable in estimating the allowable bearing capacity. In many cases the foundation design will be controlled by consideration of settlement, and greater factors of safety must be used in order to restrict the settlement to a magnitude compatible with structural requirements.

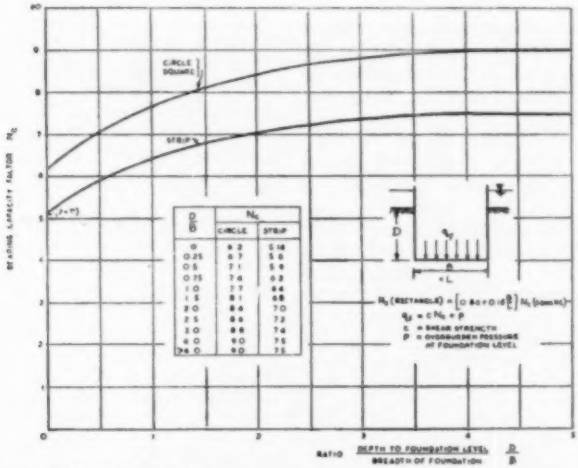
CIRCLE
SQUARE
STRIP
BEARING CAPACITY FACTOR N_c
RATIO $\frac{\text{DEPTH TO FOUNDATION LEVEL}}{\text{BREADTH OF FOUNDATION}}$ $\frac{D}{B}$
$\frac{D}{B}$ | N_c CIRCLE | N_c STRIP
0 | 6·2 | 5·14
0·25 | 6·7 | 5·6
0·5 | 7·1 | 5·9
0·75 | 7·4 | 6·2
1·0 | 7·7 | 6·4
1·5 | 8·1 | 6·8
2·0 | 8·4 | 7·0
2·5 | 8·6 | 7·2
3·0 | 8·8 | 7·4
4·0 | 9·0 | 7·5
∞ | 9·0 | 7·5
D
B
L
N_c (RECTANGLE) = $\left(0.84 + 0.16\frac{B}{L}\right) N_c$ (SQUARE)
$q_0 = cN_c + p$
c = SHEAR STRENGTH
p = OVERBURDEN PRESSURE AT FOUNDATION LEVEL

Fig. 2.—Bearing Capacity Factors for Foundations on Clay ($\phi = 0$).

Some Large Concrete Structures.

THE principal dimensions of some of the largest concrete structures in the world and in Great Britain are given in the following.

BRIDGES. — Sandö, Sweden: Span 866 ft., rise 131 ft. 3 in. (open-spandrel arch). Border Bridge, Berwick-on-Tweed: Spans 167 ft., 248 ft., 285 ft., and 361 ft. 6 in. (open-spandrel arch). Waterloo Bridge, London: Five spans 252 ft. 9 in. maximum (girder). River Marne, France: Six bridges of 243 ft. single span (prestressed concrete). (Three prestressed concrete bridges of 480 ft. span are in course of construction in Venezuela.) Sclayn Bridge, Belgium: Two continuous spans of 205 ft. 9 in. (prestressed concrete). Foot bridge, Shrewsbury: Double cantilever; main span 150 ft. (prestressed concrete; in course of construction).

BUILDINGS.—State Bank, Sao Paulo, Brazil: Height (including tower) 507 ft. (tallest reinforced concrete building in the world). Hangars, U.S.A.: Width 370 ft., length 300 ft., span of arch ribs 340 ft. Garage, Stockwell, London: Span of arch roof 194 ft., length 400 ft. (in course of construction).

PRESTRESSED CONCRETE ROOFS. — Hangar, Brussels Airport: Beams 165 ft. span. Garage, Bournemouth: Beams 150 ft. span (shell construction). Hangar, London Airport: Beams 150 ft. span (in course of construction).

SHELL ROOFS. — Hangars, Chicago: Two spans 257 ft., length 175 ft. (arch ribs and shell slab). Skating rink, Washington: Circular vault with hemispherical ends; width 103 ft., length 230 ft., rise 23 ft. Garage, Manchester: Arch roof; width 174 ft., clear span 165 ft., headroom 42 ft. Hotel, Petropolis, Brazil: Elliptical dome. Diameter 152 ft., rise 42 ft. Brynmawr, S. Wales: Rectangular domes 80 ft. by 70 ft., rise 8 ft. Domes over cylindrical tanks: Several up to 200 ft. diameter in U.S.A.

PRISMATIC ROOFS.—Airport buildings, Bristol: Spacing of columns 37 ft. 6 in. by 30 ft. Brewery, Alton: Spans 67 ft. and 32 ft.; clear working space (without columns) 65 ft. by 120 ft.

TANKS.—Staveley: 1,500,000-gallons cylindrical elevated tank; diameter 130 ft., depth 19 ft. 3 in. Glasgow: 1,000,000-gallons rectangular tank over a 1,500,000-gall. reservoir. Orleans: 1,600,000-gallons elevated rectangular reservoir (prestressed concrete).

HYPERBOLIC COOLING TOWERS.—Ellesmere Port: Height 341 ft. 6 in., diameter 260 ft. at bottom and 177 ft. at top (tallest cooling tower in the world). Hams Hall, Birmingham: Height 320 ft., diameter at bottom 220 ft.

CHIMNEYS.—El Paso (U.S.A.): Height 611 ft., internal diameter at top 14 ft., external diameter at bottom 44 ft. 10 in. (tallest chimney in the world). The chimneys at several of the new electricity generating stations in Great Britain exceed 300 ft. in height.

DAMS.—Grand Coulee, U.S.A.: Height 550 ft.; 10,500,000 cu. yd. of concrete. Shasta, U.S.A.: Height 602 ft.; 6,500,000 cu. yd. of concrete. Hoover, U.S.A.: Height 726 ft.; 4,500,000 cu. yd. of concrete. Hungry Horse, U.S.A.: Height 520 ft., length 2115 ft.; 3,000,000 cu. yd. of concrete (in course of construction). Claerwen, Wales: Height 184 ft., length 1066 ft., width 180 ft.; 400,000 cu. yd. of concrete (gravity dam). Loch Sloy, Scotland: Height 165 ft., length 1160 ft.; 200,000 cu. yd. of concrete (buttress dam).

DOCKS.—King George V Dock, Southampton: Width 135 ft. minimum, 165 ft. maximum; depth 56 ft. 6 in. minimum. Capt. Cook Dock, Sydney, Australia: Length 1177 ft., depth 50 ft. minimum; width 136 ft. 3 in. minimum, 152 ft. maximum. Sturrock Dock, Cape Town, South Africa: Length 1181 ft., depth about 46 ft. minimum; width 148 ft. maximum, 124 ft. minimum. Floating docks constructed in U.S.A.: Length 389 ft., depth 26 ft. inside 40 ft. outside, width 84 ft.

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A Large Pile Casting Plant.

LABOUR-SAVING ARRANGEMENTS.

NEW plant has recently been put into operation by Stent Precast Concrete, Ltd., for making precast reinforced concrete piles at their works at Dagenham Dock, Essex. A concrete slab, 260 ft. long and 64 ft. wide, forms the casting bed. The principal equipment comprises a batching and mixing plant, a batch carrier, a travelling distributor-gantry spanning the casting bed, and a 7-tons derrick crane with a 100-ft. jib, which are seen in this order from left to right in *Fig.* 1.

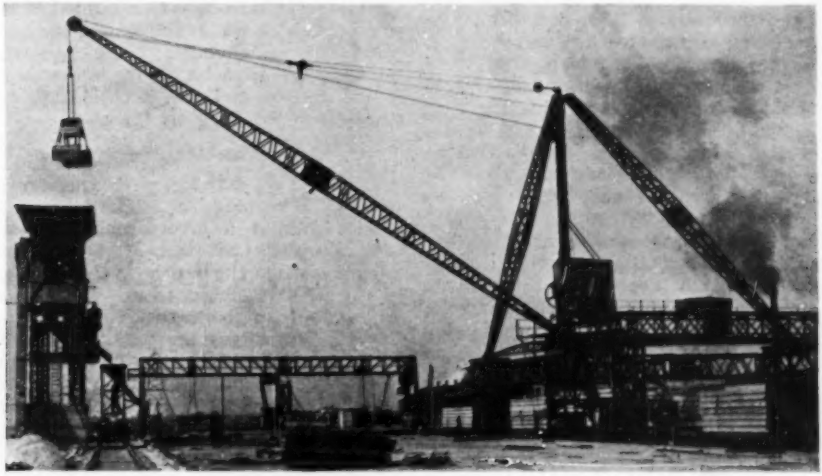

Fig. 1.—Arrangement of Pile Yard.

The Concrete Plant.

Aggregates are delivered by lorry, tipped in heaps on an 8-in. reinforced concrete pavement 20 ft. wide, and put into the steel bins of the batching plant by a 32-cu. ft. dumper-grab on the derrick crane. The batching and mixing plant, which is at about the middle of the length of the casting bed, is a standard structure erected on two concrete piers so that the chute is high enough to discharge directly into the carrier. The bins comprise three compartments and have a capacity of 60 cu. yd. of aggregate. There is also a bin to contain 1 ton of cement and which is supplied by a bucket-elevator. At present the cement is supplied in bags which are emptied into the boot of the elevator, but arrangements are being made for cement to be delivered loose and discharged directly into the boot.

The materials are automatically weigh-batched, the quantities in a 1-cu. yd. batch being generally 2050 lb. of ¾-in. gravel, 900 lb. of coarse sand, and 680 lb. of ordinary Portland cement, which are equivalent to about 1 : 1½ : 3 by weight. The water-cement ratio is 0·39. The concrete is mixed in an electrically-driven non-tilting-drum 28-cu. ft. mixer.

From the mixer, the concrete is discharged into the carrier, which is a hopper of 28 cu. ft. capacity on a steel frame carried on a four-wheel bogie propelled by a 10-h.p. petrol motor on standard-gauge track alongside the casting-bed. The carrier travels to the distributor gantry, and the concrete is discharged into the skip which travels across the gantry and from which concrete is supplied to the moulds. The carrier enables the distri-

butor to be filling moulds almost continuously, which would not be the case if it were necessary to take the gantry to the mixer to fill the skip.

To avoid high-alumina and Portland cements coming into contact with each other, a separate travelling mixing plant is provided for high-alumina cement concrete.

The Casting Bed.

The casting-bed is laid on hardcore and is 8 in. thick except at the longitudinal edges where it is 14 in. thick for a distance of 4 ft. from each edge. The slab is reinforced with two layers of ⅜-in. bars at 7½-in. centres in both directions, one layer being near the top face and the other near the bottom. The slab is in three strips, the outer strips being 22 ft. wide and the central strip 20 ft. wide, the ½-in. bitumen-filled expansion joints between the strips having steel dowel bars to prevent one strip rising relatively to another. Provision has been made to extend the length by 140 ft. The slab is laid to fall 6 in. transversely to facilitate drainage. The 75-lb. flat-bottom monorails on which the distributor gantry travels are fixed to the slab at 60 ft. centres and are about 2 ft. from the outer edges. The rail on the lower side is carried on a 13½-in. by 6-in. continuous concrete bearer through which drainage holes are provided at 6 ft. centres.

At 30 in. centres transversely, 3-in. by

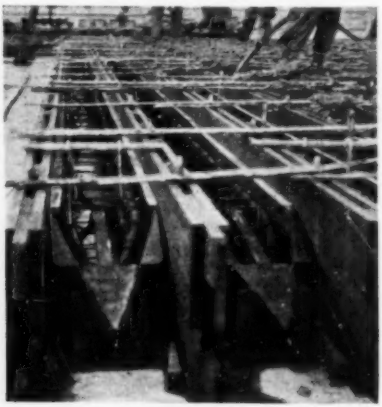

Fig. 2.—Arrangement of Shutters.

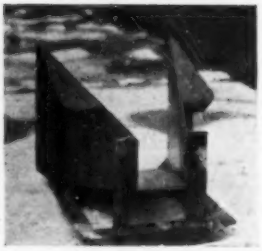

Fig. 3.—Mould for Point.

1½-in. steel channels are embedded in the top of the slab for its full length, the back face of the channels being upwards. Holes ¾ in. diameter in the channels at 1 in. centres and staggered enable the shutters to be fixed to the slab.

The Moulds.

The casting bed forms the bottom of the moulds (*Fig.* 2). The sides (or shutters) are 1-in. creosoted boards with a 1-in. cavity between. Chamfer-strips are attached at the bottom of the shutters, the depth of which is 18 in. When piles of less depth are being cast, chamfer strips for the top edges are attached to the shutters at the required height above the bed. Piles of any width that is a whole number of inches can be cast. The shutters are fixed the required distance apart by ⅞-in. steel pins placed vertically in the cavities in the shutters, the lower ends of the pins fitting into the holes in the channels in the bed. The shutters are held in position at the top by a flat steel tie which extends horizontally across two castings and connects three shutters by pins in holes in the tie projecting downwards into the cavity in the shutters. The shutters are in lengths of 10 ft.; projecting from one end of each shutter are tongues which fit into the cavity of the next length. The length of the moulds and the width of the casting bed enable piles up to 55 ft. long to be cast in this manner, but longer piles can be made by fixing the moulds along the length of the bed.

A steel plate is provided at one end of the mould for the head of the pile, and a shaped steel shutter at the other end forms the point and holds the pile-shoe (*Fig.* 3).

Before casting a pile, the wooden

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shutters, metal ends, and the concrete bed are coated with mould oil.

The reinforcement is stacked, bent, and assembled at the side of the bed opposite the batching plant. A complete skeleton for each pile is brought to the mould by the derrick crane, and is suspended in the mould by mild steel hooks attached to the flat ties. Tubes to form toggle holes are placed horizontally between the shutters.

Casting.

The moulds are filled from the 1-cu. yd. bottom-opening skip of the distributor (*Fig.* 4). At the side of the skip and travelling with it, is the operator's cab. The gantry travels along the mono-rails until the bottom of the skip is over the mould which is to be filled. The operator opens the gate at the bottom of the skip as it travels along the gantry depositing a layer of concrete in the mould, travels back depositing a second layer, and so on until the mould is filled. The concrete is compacted by electrically-operated immersion vibrators, four of which are suspended from the gantry. When the mould is full, the top face of the casting is trowelled by hand.

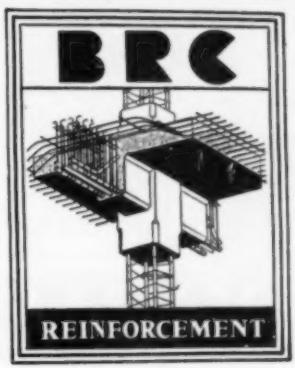
The skip and gantry are electrically operated; there are two motors for the gantry, one over each mono-rail. To ensure that each side of the gantry travels at the same speed, the motors are synchronised. Power is obtained from points at 60 ft. centres along the side of the casting bed, the cable being automatically wound or unwound on a spring-loaded drum on the gantry.

The piles are not generally removed until seven days after casting, and the method of removing them is as follows. The outer shutter of a batch of castings is removed. A bar is placed in each of the toggle holes and a shackle suspended from the derrick crane is passed under the bars. The pile is tilted slightly and immediately lowered, with the result that it moves outwards two or three inches, leaving sufficient space between the casting and the next shutter to enable a D-shackle to be attached to each bar through the toggle holes. The pile is then lifted by the crane and taken to the stacking area.

The crane, which travels on two tracks at 42 ft. centres between which is the reinforcement area, has a maximum working radius of 75 ft. and can swing through 270 deg. It can therefore serve only part of the stacking area, which is about 80 ft. wide and about 500 ft. long. Piles are restacked or loaded on lorries or railway wagons by a travelling gantry crane which is 72 ft. 6 in. between centres of rails and spans the stacking area.

Fig. 4.—Filling the Moulds.

The amount of mechanisation in this plant results in comparatively few men being employed. Apart from those concerned with the unloading of materials, assembling reinforcement, and stacking and despatching piles, there are two men on the batching plant, one man on the carrier, one on the distributor, four men on the moulds and trowelling, and a crane driver.



Bamboo Reinforcement.

THE Clemson Agricultural College of South Carolina, U.S.A., has published a report * on an investigation of the use of bamboo as reinforcement for concrete. Conclusions from the results of tests on beams reinforced with bamboo as given in the report are as follows.

Bamboo reinforcement increases the load-carrying capacity of a beam to four or five times that of an unreinforced beam of the same dimensions, but does not prevent the concrete cracking. The load-carrying capacity is greater as the amount of bamboo increases up to an optimum of 3 per cent. to 4 per cent. The safe load is two to three times that on an unreinforced beam. Unseasoned bamboo permits slightly greater loads than seasoned untreated bamboo so long as the unseasoned bamboo does not dry out while encased in the concrete. Dry untreated bamboo absorbs moisture from the concrete and the consequent swelling causes longitudinal cracks which result in a smaller load-carrying capacity, especially if the percentage of bamboo is large; this effect is less if rapid-hardening Portland cement is used.

The tensile strength of bamboo reinforcement decreases as the percentage of bamboo increases but is not affected by the cross-sectional area of a beam. The strength of the bamboo is from 8000 lb. to 10,000 lb. per square inch if the amount of reinforcement is from 3 per cent. to 4 per cent. The safe tensile strength of bamboo is therefore about 5000 lb. to 6000 lb. per square inch. If seasoned bamboo is waterproofed by asphaltic emulsion, a beam will carry a greater load than if unseasoned or untreated seasoned bamboo is used, but an excess of emulsion reduces the bond with the concrete.

The load-carrying capacity is greater

* "Bamboo Reinforcement in Portland Cement Concrete." By H. E. Glenn. Bulletin No. 4. Published by the Engineering Experiment Station, Clemson, South Carolina. 1950. (No price stated.)

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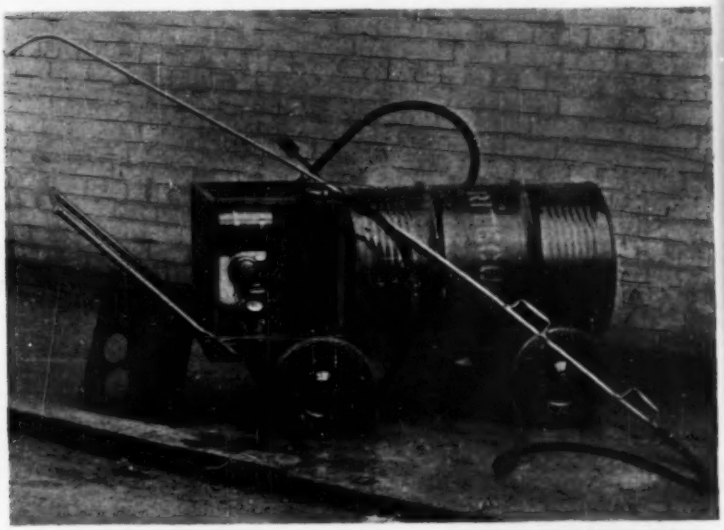

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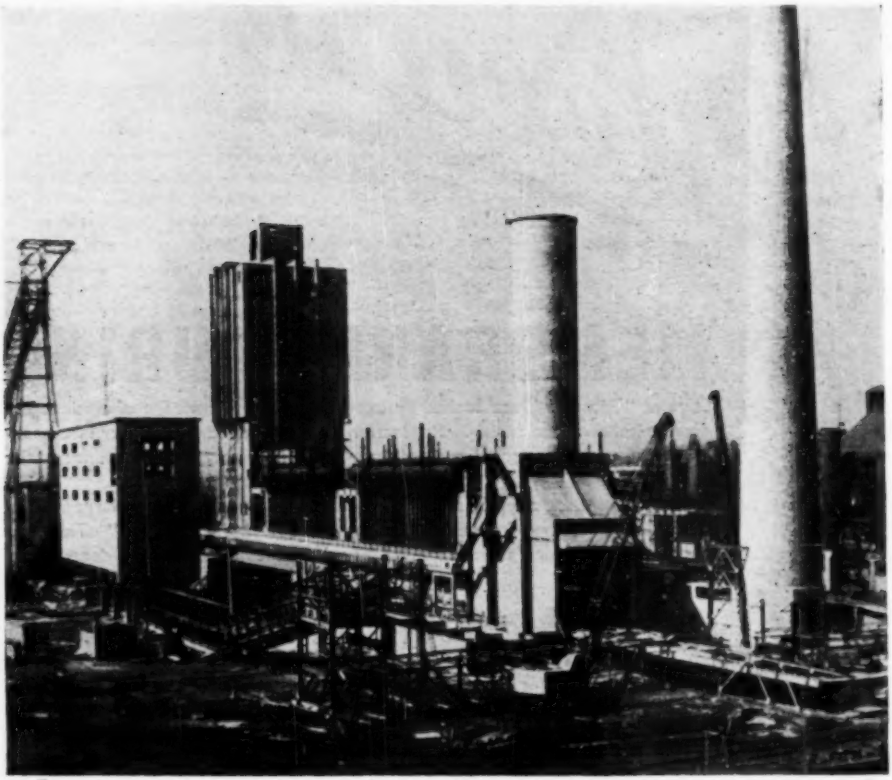

A general view of the new coking plant at the East Greenwich Gasworks, designed and constructed by Messrs. Simon-Carves, Ltd., for the South Eastern Gas Board. All the structures shown are supported on Simplex Concrete Cast-in-Situ Piles.

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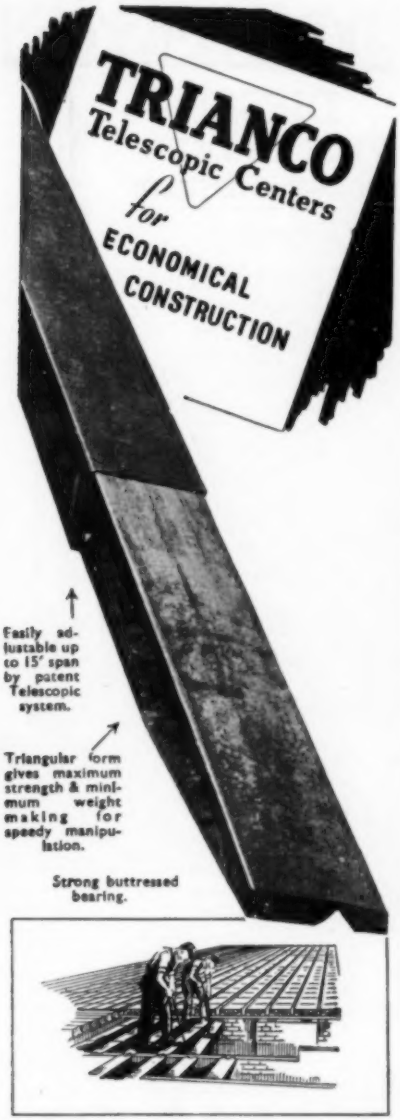

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if unseasoned bamboo is split longitudinally, and still greater if split seasoned bamboo treated with emulsion is used, but if split untreated bamboo of large diameter is used the load-carrying capacity is virtually destroyed because of longitudinal cracking. Unseasoned bamboo may dry and shrink while encased in concrete, and in time the bond between the bamboo and concrete is reduced and consequently the load-carrying capacity is reduced.

The greater the strength of the concrete, the greater the load-carrying capacity of a beam. The load-carrying capacity is also increased if split bamboo is provided to resist diagonal tensile forces resulting from the shearing force. Failure of beams reinforced with bamboo is generally due to failure in diagonal tension even though reinforcement for diagonal tension is provided. The deflection of a beam is proportional to the load until the concrete cracks, when a greater increase in deflection occurs and probably denotes partial failure of bond; subsequently the deflection is again proportional to the load up to failure, but the rate of deflection is greater than before cracking occurs.

The behaviour of tee-beams reinforced with bamboo is similar to that of rectangular beams, but the strength of a tee-beam is no greater than that of a rectangular beam of the same effective depth if the breadth of the rib of the tee is the same as the breadth of the rectangle.

The following recommendations are given in the report.

In important beams whole stems of green unseasoned bamboo are not recommended, but if such stems do not exceed $\frac{3}{4}$ in. diameter they may be used in slabs and secondary beams. Bamboo in beams should, where possible, be cut and allowed to season for three to four weeks before use, but stems cut in the spring or early summer are not recommended. Stems should be at least three years old before cutting. When well-seasoned whole or split stems are used in important beams, waterproofing is recommended. In slabs and secondary beams unseasoned split stems can be used if not exceeding $\frac{3}{4}$ in. wide and if rapid-hardening Portland cement is used; the space between adjacent laths and between layers should be $1\frac{1}{2}$ in. to 2 in.

Diagonal tension should be resisted by vertical laths of split bamboo and by

bending up the longitudinal reinforcement.

The strength of a beam is adversely affected if the stems are placed too closely. If there are several layers, the top layer should not be too close to the neutral axis otherwise there may be insufficient concrete to resist horizontal shearing forces. Stems parallel to each other should be placed so that the wide and narrow ends alternate so as to ensure a fairly uniform cross-sectional area throughout the length of the beam and an increased bond due to the wedging effect.

If the deflection must not exceed $\frac{1}{850}$ of the span, the calculated tensile stress in the bamboo should not exceed 4000 lb. per square inch; this assures also a high factor of safety against failure in bending. The design of beams reinforced with bamboo can be the same as that for beams reinforced with steel bars. The modulus of elasticity of bamboo in tension varies with the species and is from 2,000,000 lb. to 4,500,000 lb. per square inch. In the design of a tee-beam the flange should be ignored. Bamboo is about four times stronger in tension than in compression; the moduli of elasticity in compression and tension are about the same.

Several experimental buildings were erected in concrete reinforced with bamboo, and the following are the principal points from the reports of their condition after $3\frac{1}{2}$ to 5 years.

The footings, columns, beams, and slabs of the first structure up to the level of the floor appeared to be in good condition although loads in excess of the design load had been imposed. Small non-progressive cracks occurred in some beams but none in the floor slab. The beams supporting the roof were unsatisfactory, since cracking commenced a short time after construction and shores had to be placed when the building had been up about six months. No cracks occurred in the roof slab. The failure is attributed to the mixture of the concrete in the roof beams being 1 : $2\frac{1}{4}$: $4\frac{1}{2}$; the several layers of bamboo not being tied together; the bamboo not being prevented from floating when the concrete was placed; due to high temperatures, the unseasoned bamboo was quickly dried and the bond with the concrete was probably ineffective.

Cracks occurred in a three-story building a few months after erection, but did not seem to impair the strength. The

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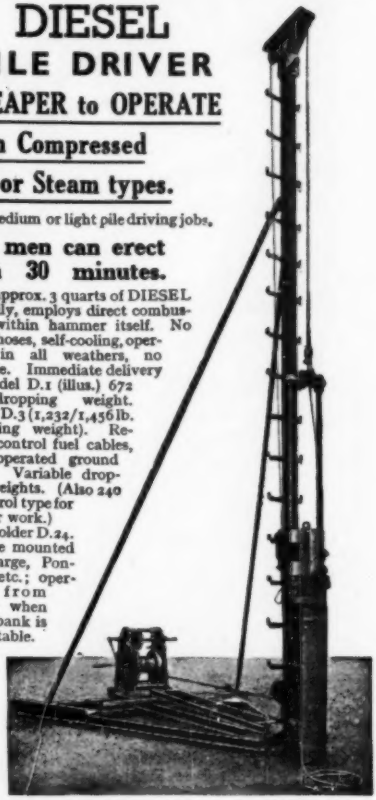

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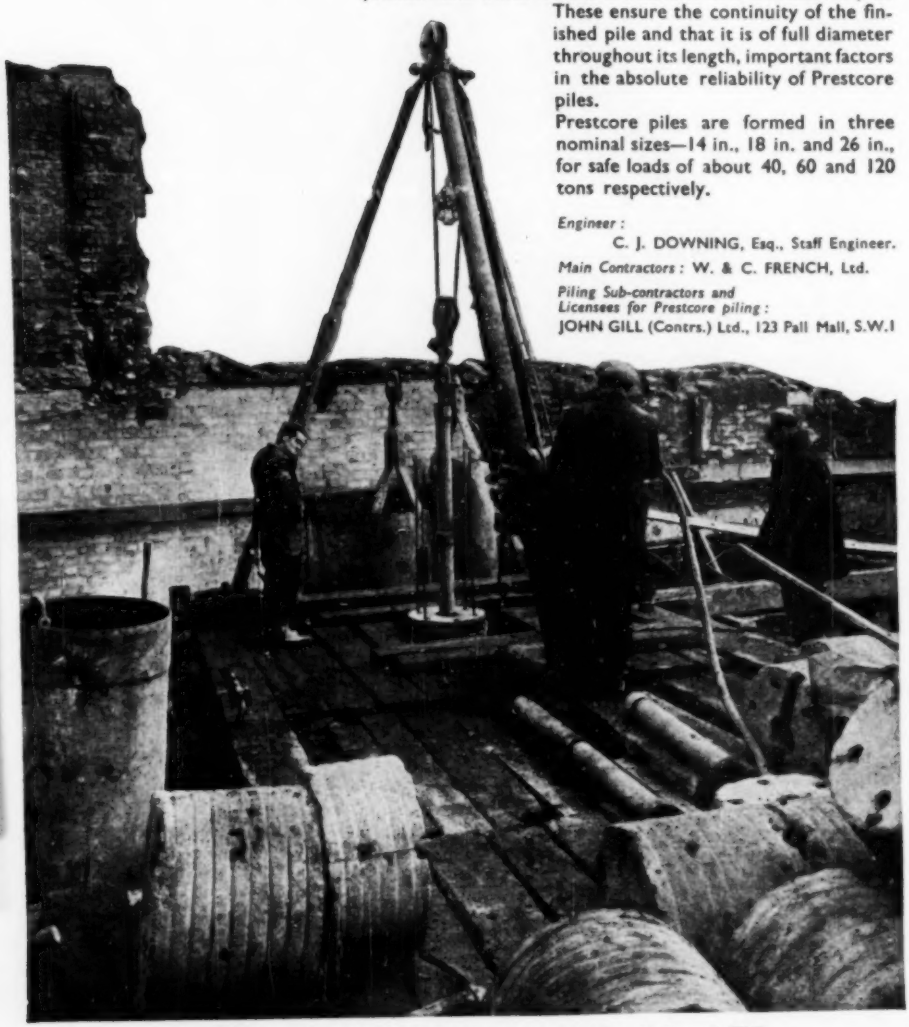

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deflections were small. After five years, no additional cracks were noticed nor had the earlier cracks extended much.

The floor and roof slabs and structural frame of a farmhouse were constructed in bamboo-reinforced concrete, and after 4½ years the floor slab and supports showed no sign of failure although there were small cracks in some beams. The roof slab has a length of more than 100 ft. without contraction joints but there are no cracks, but the roof beams cracked so seriously within four months of construction that additional supports were provided; some of the bamboo was exposed when the shuttering was removed.

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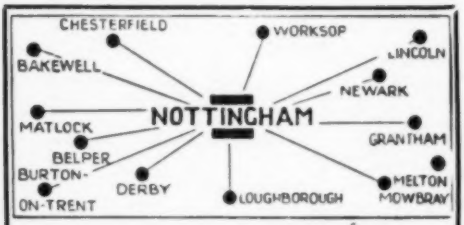
[A review of German and Italian tests of beams reinforced with bamboo was given in this journal for March, 1941, pages 109 to 115.—ED.]



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SITUATION VACANT. Consulting engineers, Westminster, have vacancy for senior reinforced concrete detailer on their permanent staff. Commencing salary £850 p.a. Applicants must have had not less than five years' experience in a good reinforced concrete drawing office in a reasonably responsible position. Applications, which will be treated in confidence, stating age and experience, to be sent to BOX 2495, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Junior and senior draughtsmen, with good experience in all branches of reinforced concrete, wanted for consulting structural engineer's office. Experience in structural steelwork an advantage. Salary according to age and experience. Please apply in writing to F. J. SAMUELY, 8 Hamilton Place, London, W.1.

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(*Continued on p.* 386.)

Miscellaneous Advertisements

(*Continued from page 385*)

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co. Ltd. require several qualified designers with specialist experience for their Stafford, London, Bristol, Glasgow, Dublin and Newcastle-on-Tyne offices. Five-day week and staff pension scheme. Apply to B.R.C. ENGINEERING CO. LTD., Stafford.

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SITUATION VACANT. NORTH THAMES GAS BOARD. SENIOR DRAUGHTSMAN required, experienced in the design and detailing of reinforced concrete structures and general civil engineering work. The appointment is of a permanent nature and pension arrangements will be discussed with short-list candidates. Starting salary will be within the range of £650–£800 per annum, depending upon age, qualifications, and experience. Applications, giving age and full particulars, should be sent to the STAFF CONTROLLER, NORTH THAMES GAS BOARD, 30 Kensington Church Street, London, W.8, quoting reference number 91.

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SITUATION VACANT. Wanted by old-established British firm in Pakistan, a qualified civil engineer with practical experience of reinforced concrete construction and design work. Terms: three years' agreement, free passage and attractive salary according to experience. Single man preferred. Write, stating full particulars, to Box "NV/72" c/o 95, Bishopsgate, London, E.C.2.

SITUATION VACANT. Reinforced concrete engineers require the services of a competent engineer for site inspection work. The applicants should possess considerable experience as resident engineers or in similar supervising capacity in the erection of reinforced concrete structures. Some design experience desirable. Visits to sites in all parts of the country will be entailed. Headquarters in London. The post is permanent, and a superannuation and profit-sharing scheme is in operation. Write, giving details, to BOX C.E.576 at 191 Gresham House, E.C.2.

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